DESIGN AND EXECUTIVE ASPECTS OF THE CONSTRUCTION OF THE LINE 1 OF THE NAPLES METRO

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ABSTRACT

The lower stretch of Line 1 of the Naples subway is at present under construction. It is characterized by the relatively great depth of the tunnels, which are located within the base formation of Yellow Neapolitan Tuff, up to 30 m below the water table. The geological framework is briefly presented. The design criteria and construction methods adopted for the stations and the running tunnels (which are bored by means of two EPB shields) are then reviewed, stressing the most relevant aspects of the project.

1. BACKGROUND

The design of line 1 of the Naples underground commenced at the end of the 1970s when the possibility of connecting the hilly district on the outskirts of the city with the centre was evaluated. In 1997, Line 1 became part of an integrated rail transport network with a ring route to connect the airport, the newly built up peripheral districts, the city centre and the port district with the main railway station. The section between the expanding urbanised area in the North of the city and Piazza Dante in the city centre is currently in service. The lower section (Dante – Centro Direzionale Nord) is currently under construction and will link important districts of the city running from Vomero and via Toledo to Piazza Municipio touching on the Maschio Agioino wall, Corso Umberto and Stazione Centrale and then the Capodichino Airport (figure 1).

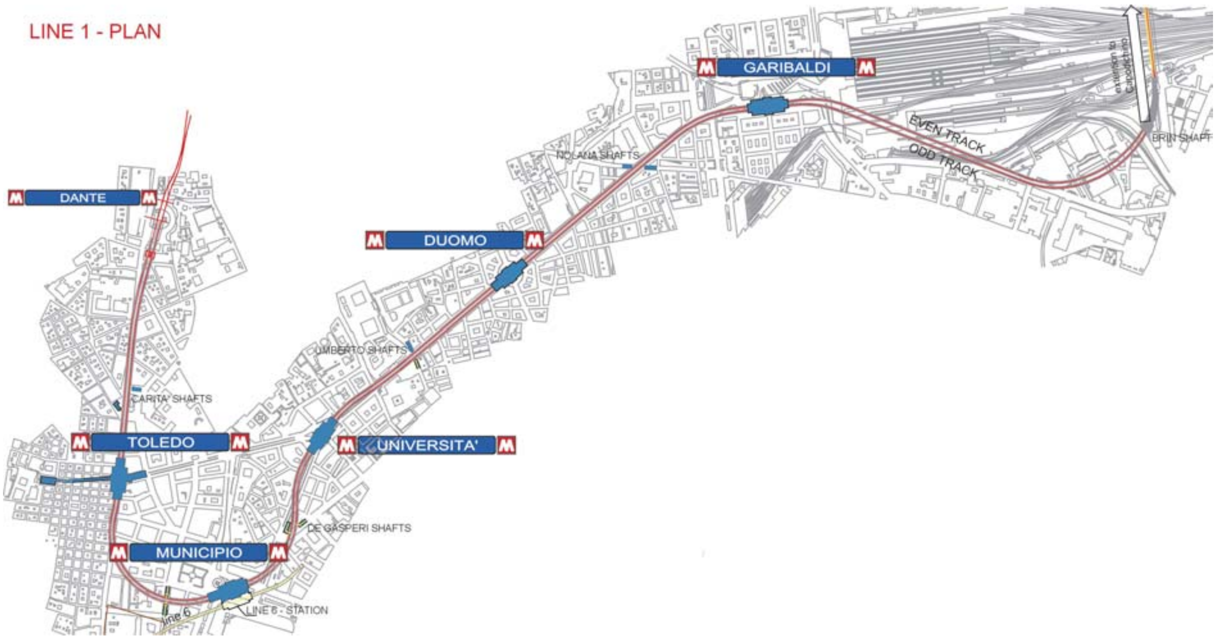

Figure 1 Route of the lower section of line 1 (under construction)

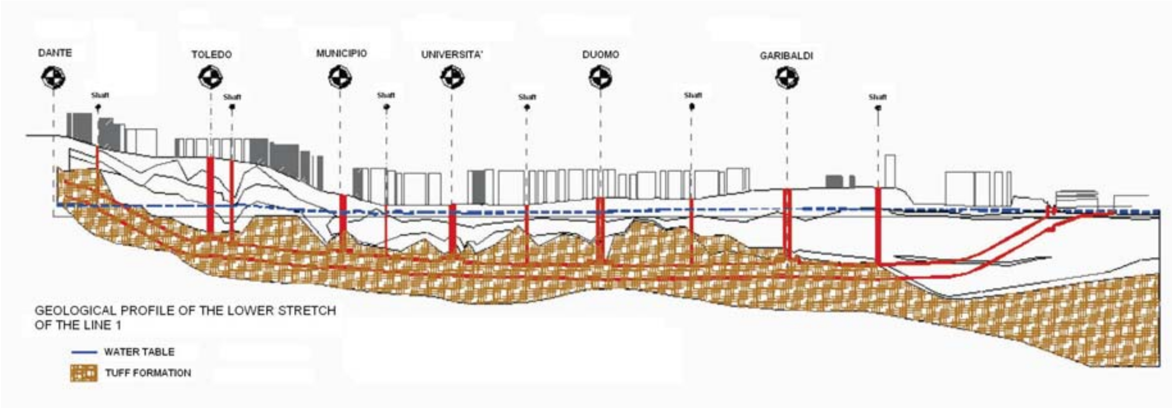

Figure 2 – Geological profile of the lower section of line 1

The railway layout consists of two line tunnels side by side with an inner diameter of 5.85 m and a distance between centre lines of approximately 11 m. and of five stations (Toledo, Municipio, Università, Duomo and Garibaldi). The stations consist of a cut-and-cover excavated central shaft with a cross section of 45 x 20 m. excavated between reinforced concrete diaphragm walls placed using hydro-cutters. The 4 station tunnels and the 4 access stairway tunnel driven using bored tunnelling methods lead off from the shaft. The design of the underground works had to address and solve problems typical of tunnelling in an urban area such as the need to limit surface settlement and maintain safety under a shallow overburden with high water heads on the basis of the specific conditions of the Neapolitan terrain. The problems were solved by employing some of the most advanced and reliable construction technologies available.

2. GEOLOGICAL AND GEOTECHNICAL CONDITIONS

The stratigraphy of the ground in the section between piazza Dante and Garibaldi Station, below the first level of anthropic deposits rich in archaeological remains with a thickness varying up to a maximum of 8-10 m., consists of loose materials such as sands and pozzolans with a thickness of 10-15 m. over bedrock consisting of Yellow Neapolitan Tuffs, a soft and porous volcanic rock. After Piazza Dante the route passes mainly through sandy silty deposits (figure 2).
The pozzolans and tuffs have a chaotic structure with an ash-vitric matrix and are often separated by a transition stratum which is generally more porous named "cappellaccio" or Neapolitan Yellow Tuff. The stonier part of the tuff is fractured subvertically along the pathways taken by the gas with which it was impregnated escaping, a phenomenon known as "scarpine", formed following the slow cooling of the pyroclastic mass.
The free surface of the water table below Via Toledo runs on average at an altitude of 4.5 m. above sea level and tends to fall gradually towards the coastal areas (2.5 m. a.s.l. in Piazza Garibaldi).
The entire route, fully below sea level, is therefore well under the water table sometimes more than 20 m. below it.
This situation, together with the presence of randomly scattered cooling factures inside the tuff could be potentially dangerous, because the fractures would constitute pathways along which water would tend to flow into the tunnels under construction and connect the excavation with the loose ground above. Permeability of one to two orders of magnitude greater than that of the tufaceous rock mass could arise if these fractures opened and filled. The high velocity of the flows triggered by excavation could trigger entrainment and the flow of solid materials from the loose upper strata with a higher risk of side-effects at ground level.
In the section which passes through the loose deposits behind Central Station, the typical problems of an unstable rock mass are encountered in excavation work. The sands are loose or poorly compacted and under high water heads, subject also therefore to the destabilising action of flows of water towards

the face induced by the relaxation of the face-advance core. Given the weak gradient between the level ground inland and that of the coast line, the water moves with a reduced permeation velocity in the surface ground which has a higher degree of permeability (10^{-3} – 10^{-5} cm/sec) than the underlying stratum of tuff. The primary permeability of the latter is low but in reality the tests conducted and the work in progress have demonstrated that there is a higher secondary permeability comparable to that of the surface stratum, which is strictly connected with the degree of fissuring.

3. DESIGN APPROACH

The approach adopted is based on the ADECO-RS approach by Lunardi (2000).
In the *survey phase,* the focus was set not only on the characterisation of the ground, but also on the urban context understood as the geometrical definition of the potential interferences, even indirect, with the pre-existing structures.
On the basis of the results of this characterisation, the deformation response of the core-face and of the cavity was analysed in the *diagnosis phase*, for the purpose of identifying the types of deformation expected, the manifestations of instability and the loads mobilised by excavation. The importance which is placed in that phase on assessing possible settlement phenomena induced at street level, a potential risk factor for pre-existing buildings, is clear.
The stabilisation operations and the advance methods needed to perform excavation correctly and to prevent seepage action which would have put the excavation in contact with the surface were identified in the therapy phase. In this specific case this translated into: selecting the type of TBM for the line tunnels and the specific action to be taken to confine water in traditional excavations; defining the advance methods; identifying and deciding the magnitude of pre-confinement and of ground improvement in the advance core; identifying solutions for conditions in specific points where there was interference with pre-existing buildings (protection works).

3.1 The ADECO-RS approach: evaluations and basic considerations

The design of the bored tunnels was conducted according to the principles of the ADECO-RS approach Lunardi (2000), which has the objective of assuring the formation of an arch effect close to the profile of the excavation by controlling the deformation response of the rock mass to excavation of the tunnel.
The very large bibliography may be consulted for a description of that approach, which is in common use in Italy and forms part of the specifications of all the public administrations and contract awarding bodies.
We wish, however, to underline its specific characteristic of providing support not only during the actual design phases themselves (design stage), but also during the construction stage. The interpretation of the measurements recorded by the monitoring and control system installed (for the deformation response), does in fact play a fundamental role in making it possible to adjust and balance the stabilisation and advance systems employed, in order to ensure that the tunnel is constructed to the design specifications (by comparing the measured deformations with those predicted in the design stage).
In this specific case the analyses conducted in the diagnosis phase led to the definition of two distinct situations, according to whether the tunnel lay within the tufaceous formation or the sandy deposits.
In the latter case the nature of the ground and the surrounding conditions resulted in the prediction of "unstable" face behaviour (category C): in the absence of confinement action, the deformation phenomena at the face develops rapidly into the elastic-plastic range until failure conditions are reached with the failure of the face without an arch-effect being created around the excavation and the consequent instability of the cavity also. In order to guarantee stability, action must be taken to confine the face and the cavity that is capable of exerting pressure of a great enough magnitude to maintain the conditions of the ground practically undisturbed in terms of stress states and deformation. It is possible to minimise the "losses of volume" during excavation in this manner, and therefore prevent the ground from loosening around the cavity and the resulting side-effects on the surface.
Although no particular problems connected with significant deformation phenomena were encountered passing through the tufaceous formation, confinement action had to be exerted in the same way at the

face and around the cavity to limit risks connected with the attraction of water exerted by the decompression that was induced by excavating under water table.
For the line tunnels, the therapy phase translated into selecting the most appropriate type of TBM to exert those actions, and that is TBMs capable of balancing the earth pressure at the face. This confinement action had to be exerted:

- at the face by maintaining a predetermined confine pressure. Once the ground enters the machine through openings in the cutter head, it is in fact held under pressure in the excavation chamber. It must present appropriate characteristics of plasticity and liquidity and low permeability to be used as a support fluid. That is why it must be conditioned using additives such as polyurethane foams and/or bentonite. The thrust of the machine exerted on the face by jacks which push against the prefabricated concrete segments of the lining compacts the material in the excavation chamber (internal pressure), to balance the pressures of the ground and water (external pressure). The "balanced" condition is reached when the material in the excavation chamber can no longer be compacted further by the external pressure. In this case, an increase in the internal pressure in the excavation chamber produces excess pressure at the face, which can cause the ground to rise at street level, given the magnitude of the overburden. If, however, the internal pressure is reduced, this produces loss of volume and generates settlements at street level. In practice the internal pressure in the excavation chamber is determined by controlling firstly the speed of advance of the machine and the quantity of material excavated that is extracted from the excavation chamber through the screw conveyor and secondly the plasticity and fluidity of the material excavated (type of conditioning). When a greater volume of material is extracted than is coming in (speed of the machine), the pressure in the excavation chamber falls and vice versa when less is extracted the pressure increases. The pressure in the excavation chamber is measured by means of sensors installed at different levels in the chamber which are read remotely in the control cabin.
- around the cavity. This action must be exerted ahead of the TBM shield and afterwards by the lining ring of prefabricated segments made solid with the ground by backfilling the ring of hollow space between the extrados of the ring and the profile of the excavation with cement grout injected under pressure from the tail of the shield. Long term confinement action is exerted by the reinforced concrete ring of the lining which supports the pressure from the ground and the water with adequate margins of safety. The diameter of the front of the shield is slightly greater than the end of it (it is conical), which reduces the risk of it becoming stuck during shield advance. Convergence of the diameter of around 20 mm. is therefore usually allowed in that zone. Since in all probability the ground is unable to maintain the original profile of the excavation, a part of that convergence is transformed into a loss of volume around the cavity and this must be taken into account when surface settlements are estimated. Further losses of volume around the cavity should not occur since the backfill behind the ring of segments is performed from the tail of the machine immediately after the segments are placed (rings of metal brushes continuously injected with high density grease prevent the injected grout from flowing back into the shield) without allowing any increase in the convergence of the cavity. On the contrary the introduction of the backfill grout balances the pressure exerted by the ground and the water and if it is then increased appropriately, it can exert confinement action on the cavity itself. In any case, given the delicacy of the operation, systematic monitoring must be performed during excavation in order to verify whether a balance has been reached between the volume to be filled and the quantity of cement grout injected.

In order to guarantee the maximum impermeability around the excavation for the platform stations and the access stairways, the most appropriate technologies were selected from those available to employ in the heterogeneous and complex rock mass found in the ground below Naples. The tuff was found extremely difficult to inject because of its intrinsically low permeability and the presence, not detected beforehand, of discontinuities in permeability up to two orders of magnitude greater than expected which could raise secondary permeability drastically. The answer to the problem was found by freezing the water around the tunnels to be driven by using a system of longitudinal freezer pipes arranged around the profile of the excavation. The thermal inertia of the ground combined with the circulation of water, even if this was weak, meant that this work had to be performed using liquid nitrogen until a wall of ice was obtained at least 1 m. thick with a temperature or around – 10° C.. Excavation and the casting of the final linings were then performed during the “maintenance” period, guaranteed indirectly by circulating a brine consisting of water and calcium chloride at a temperature of less than – 30° C.. Drilling the holes to house the freezer pipes, performed from the station shafts for lengths of even greater than 60 m. was done using sophisticated guidance systems and mechanical “preventers” to prevent the dangerous entrance of water into the shaft.

3.2 Line tunnels

From a design viewpoint, the vertical geometry of the line tunnels was such that they ran for almost their entire length through the stony yellow tuff formation characterised by good mechanical properties. The need to limit effects on the surface, minimise the impact on the urban fabric and prevent the infiltration of water from ahead of the tunnel already described meant that shallow overburden cut and cover methods were ruled out, because a large part of the route ran parallel to the coast line below districts with a high density of motor traffic and large stonework and brickwork buildings with shallow foundations. Furthermore the tunnels will run completely below the water table except for an initial section located immediately after Piazza Dante. The maximum water head (approximately 30 m. at track level as already mentioned) is under Piazza Garibaldi.
Even if the permeability of the tuff is low (10^{-4} – 10^{-5} cm/s), in those sections where the rock overburden is shallower, or where sub vertical fractures (“scarpine”) are present in the rock, there is a real possibility of serious flooding with water flowing in concentrated in points through the loose ground in the overburden, dragging solid material with it and causing serious surface subsidence. The phenomenon is sadly well known in Naples, the result of cavities scattered in the subsoil and the ease with which the loose pyroclastic materials erode.
Considerations such as

- the geometry of the alignment and tunnel section type;
- the surrounding conditions (environmental constraints);
- the geotechnical and hydgrogeological conditions;

where the objective is to construct a tunnel in safety under difficult conditions reducing the risk of direct and/or indirect interference with pre-existing structures, led to the decision to employ two closed shields (one for each bore) with a diameter of 6.75 m (photo 1) capable of exerting sufficient pressure against the face to contain extrusion and prevent water from entering. The immediate lining of the walls of the excavation is performed using an impermeable lining of prefabricated concrete segments erected inside the shield and backfilled with grout outside the shield using extruded concrete.

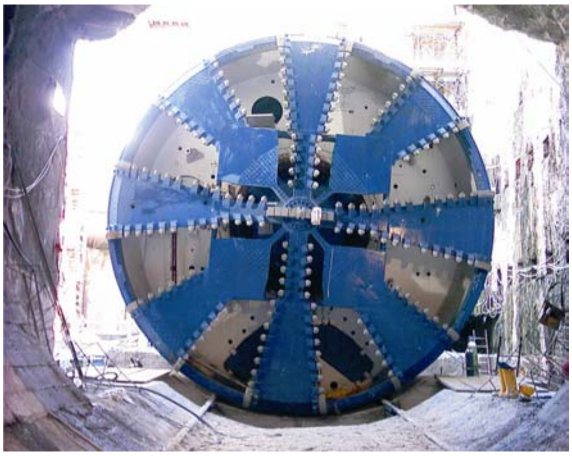

Photo 1. View of the TBM cutter head

Photo 2. Line tunnel driven using mechanised excavation

The technology selected was that of EPB (Earth Pressure Balanced Shields) rather than hydroshields because of the greater flexibility of the EPB systems in terms of logistics and excavation operations, in the light of the potential offered today by systems for conditioning the materials to be excavated, which has broadened the working range of EPB TBMs in terms of the soil or soft rock to be excavated.

From a strictly design viewpoint, the critical working conditions, significant for the correct design of the TBM, take concrete form in terms of the maximum working pressure that it must be able to exert at the face. Assessments of the deformation behaviour of the rock mass and of the level of the water table above the tunnel to be driven, led to the hypothesis of a maximum working pressure of 4.5 bar (inclusive of an appropriate safety coefficient).

The actual design of the TBM by the manufacturers (Herrenknecht), therefore led to the construction of a tunnelling machine with the following characteristics:

– diameter of excavation 6.75 m.;
– maximum torque 7,300 kNm.;
– maximum velocity of rotation 3.5 rpm;
– rated wattage 1,200 kW;
– maximum thrust 41,000 kN (19 cylinders dividing into 6 groups);
– erector with vacuum system;
– conditioning through 9 injection points (5 on the wheel and 4 in the excavation chamber);
– backfilling through 4 independent injection lines, with a dual component (grout+accelerator) system and automatic control of pressures and volumes.

This method of operation guaranteed effective and immediate confinement of all the excavated surfaces of the tunnel during construction:

- at the face by counter balancing the hydrostatic pressure and the earth pressure on the head of the shield;
- around the cavity by erecting an impermeable lining of prefabricated segments made immediately active by using extruded concrete to backfill the space which inevitably remains between the ground and the segments;

The method minimised the "losses of volume" during tunnel advance, eliminated the risk of potential flooding and last but not least provided full control of surface subsidence.

Important advantages are also obtained from an environmental viewpoint:

- the water table is not affected in any way at all because the pressure of the water at the face is constantly counterbalanced by the hydrostatic confinement systems of the machine;
- the materials employed are not polluting. The ground excavated is mixed with water and special additives and then conveyed outside the tunnel where, because of its intrinsic properties, once separated from other elements, it is used in the construction of other civil works (embankments, raised areas, etc.).

3.3 The final lining in prefabricated segments: the universal ring

The final lining of the line tunnels is placed directly from the TBM by means of an erector installed inside the shield. This lining, consisting of an assembly of prefabricated segments produced off site, performs the following functions: it confines long term loads during the useful life of the tunnel, counters transitory actions when the machine is thrust forward, it provides water proofing and follows the design alignment of the tunnel.

A "universal ring" was chosen for the geometry with an obliquely truncated shape which allows for great flexibility in adapting to the vertical and horizontal geometry of the alignment. The basic concept is that each ring (consisting of a number of segments) should be conical so that it can follow the direction of the alignment by successive approximations by rotating it appropriately with respect to the previous ring installed which is done simply by assembling the different rings in the right sequence each time. Water proofing is guaranteed by means of seals fitted continuously along the outside of the segments.

The salient characteristics of the lining employed are as follows:

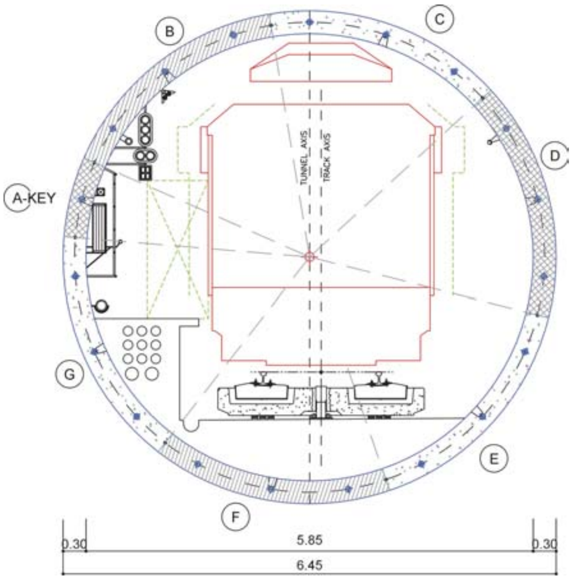

Figure 3. Running tunnel section type.

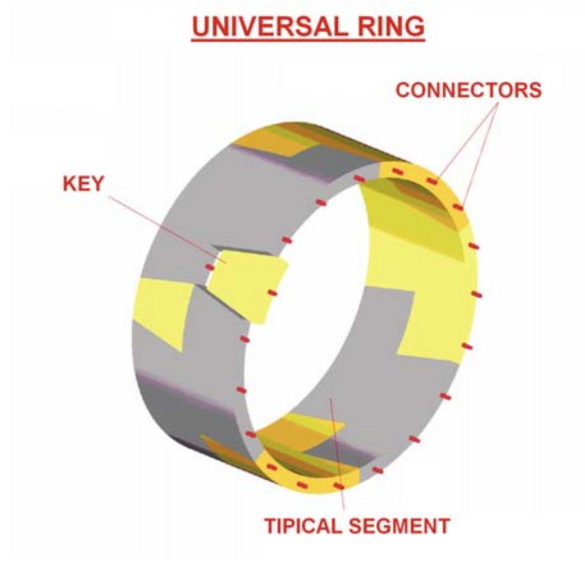

Figure 4. The Universal Ring.

outer diameter of segment ring:	6,450 mm.;
inner diameter of segment ring:	5,850 mm.;
thickness of segments:	300 mm.;
minimum actual radius of curvature:	180 m.;
design radius:	150 m.;
average length of segment ring:	1,200 mm;
minimum length of segment ring:	1,174 mm.;
maximum length of segment ring:	1,226 mm.;
number of segments per ring:	6 + key;
average ring weight:	18 t;
average segment weight:	3 t;
concrete:	R_{ck} 45 MPa;
reinforcement:	FeB44k steel, 100 kg/m^3 approx.

3.4 Shafts, station tunnels and stairway access tunnels

Space on the surface is generally limited and problems with traffic circulation resulted in the design of a standard structure common to all stations.

Photo 3. View of the Garibaldi station shaft.

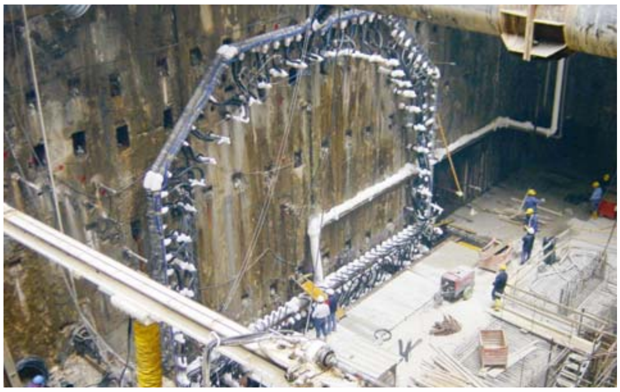

Photo 4. Platform tunnels: start of the freezing stage

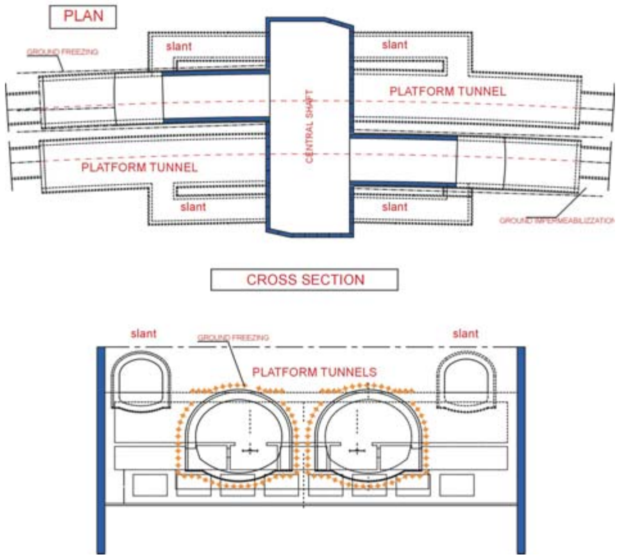

Figure 5. Design layout for the station tunnels.

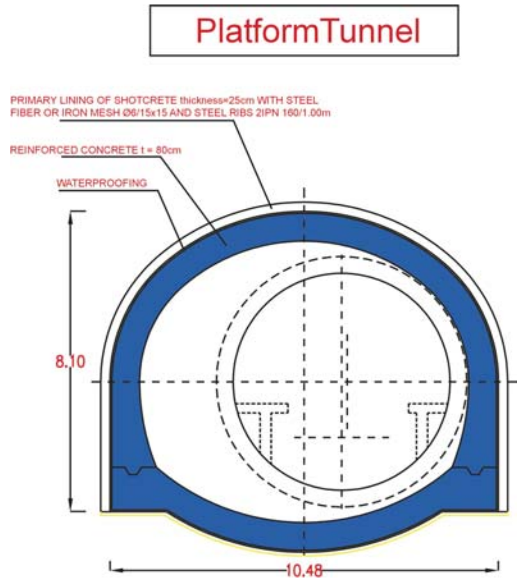

Figure 6. Cross-section of a station tunnel.

The designs consisted basically of rectangular shafts, 20 x 45 m. approx., driven down to a depth of 35 – 50 m. and centred over the running tunnels (Fig. 5 and photo 3), except for the Toledo station for which the shaft was placed to the side of the line tunnel since it was impossible to centre it over the alignment.

The overburdens for the platform tunnels are between 25 m. and 30 m. and are connected to the shafts by the tunnels for the stairways for passengers (4 for each station) which run parallel to the line tunnels.

These tunnels were constructed with a larger cross section (55 sq. m. of inner diameter) to house the station platforms and run for around fifty metres in both directions (figure 6).

Given the poor geomechanical properties of the ground to be excavated and the problems of advancing under the water table, it was essential, before the start of excavation, to design a confinement perimeter structure for the construction of the shafts which would give guarantees against inflows of water. It consisted of flat panel diaphragm walls excavated using a hydro-cutter. The perimeter diaphragms were braced by the concrete slabs of the station floors which were constructed as the excavation reached the relative depths.

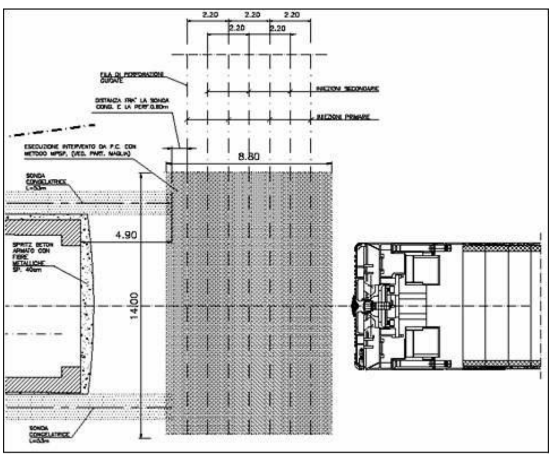

Figure 7. Buffer constructed from ground level – Longitudinal section

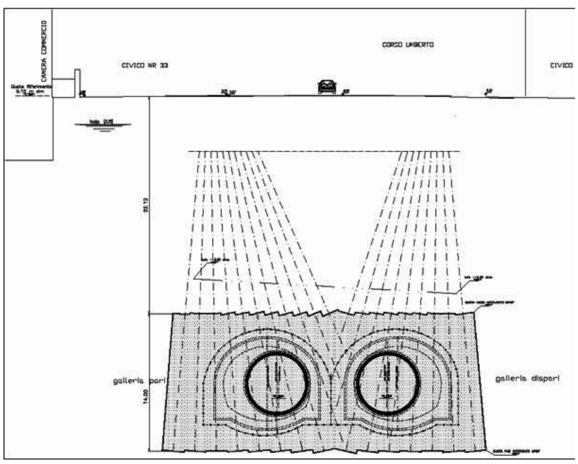

Figure 8. Buffer constructed from ground level – Cross-section

Photo 5. Freezing using nitrogen

Photo 6. Excavation of the platform tunnels under freezing conditions

Finally the platform (photo 4) and stairway access (with a cross section of 33 m^2 approx.) tunnels were constructed from the shafts.
The crown of the platform tunnels generally passed through materials in a transition zone where the lower tuff stratum comes into contact with the overlying pozzolans and/or sands, when it did not actually pass directly through the latter.
The presence of groups of vertical fractures resulting from freezing in the rock stratum, potentially capable of creating a path for water to pass between the tunnels, inside the tuff, and the upper strata consisting of loose materials, posed the problems of confining the water and of the possible entrainment of solids around the works to be constructed. Furthermore, analysis of the data acquired showed high secondary permeability of the rock mass and therefore great sensitivity of the water table to operations potentially capable of creating infiltrations of water and disturbance to the ground frozen for excavation operations.
The following action was therefore taken to prior to excavation in order to guarantee safety:

1) a buffer was constructed of approximately 9 m. (see figures 7 and 8), consisting of a series of holes drilled from ground level before freezing was activated, which separated the works from those of driving the tunnels. They were injected with cement and chemical mixes using MPSP (Multi Packer Sleeved Pipe) technology.

   This technology, typically employed for fractured rock, involves isolating the active valves by using a double barrier bag system. With this method each section drilled is confined above and below by two barrier bags set at intervals of approximately 2.5 m.. As compared with other technologies, this has the advantage of not requiring costly redrilling of injected sections and it in any case performs a check on the diffusion of the mix with in the rock mass.

   Without the presence of a transverse buffer to close off the frozen ground, flows of water could have been triggered running co-axial to the tunnel itself.

   This dangerous event could have produced:

   a. the transport of fine material and/or consolidation phenomena in the upper strata consisting of sands and pozzolans;

Photo 7. Platform tunnel – Garibaldi Station – Traditional excavation in frozen ground

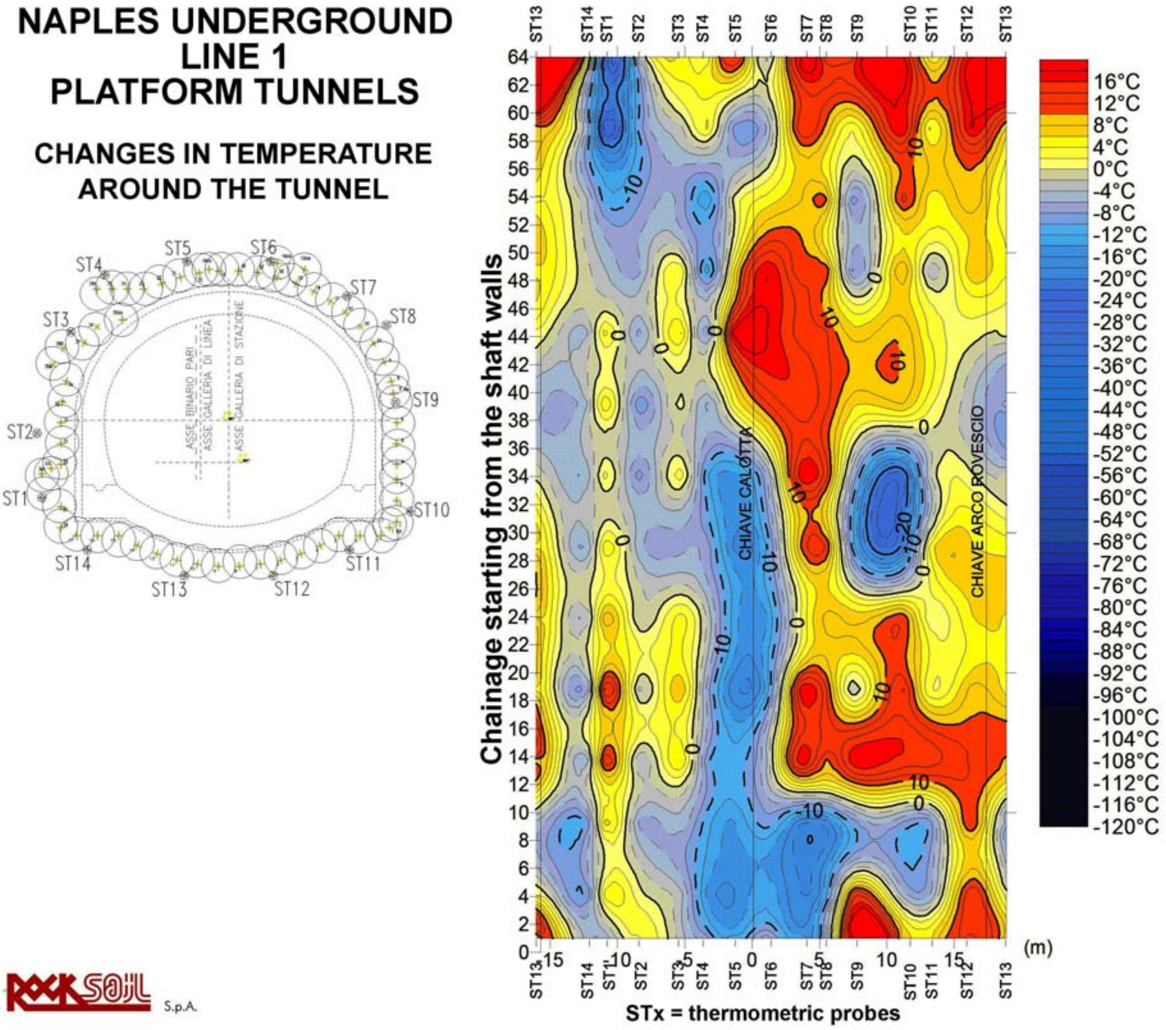

Figure 9. Two dimensional thermometric diagrams.

b. loosening of the frozen ground around the tunnel;
c. a slowdown in excavation and lining operations for station tunnels. In this respect the excavation of the first platform tunnel showed how this can occur without any particular stress-strain phenomena sufficient to have to halt tunnel advance to cast the tunnel invert the side walls and the crown at close distances from the face. It was however necessary in two cases to stop tunnel advance to cast the final lining in order to protect the surrounding wall of ice and confine the ground in the presence of progressive thawing phenomena;

2) ground improvement and impermeabilisation was performed by creating a shell of ground of adequate thickness around the future cavities. This was achieved by using a freezing mix of nitrogen and brine with the refrigerant fluid circulating in pipes appropriately positioned in the ground (see photos 5 and 6) at very low temperatures (-195° C for the nitrogen and -35° C for the brine). With the mixed system, freezing is performed firstly by the liquid nitrogen and afterwards, during the maintenance phase, by the passage of brine once the ground has been frozen right around the whole tunnel. The need to reach the required temperature of -10°C at a distance of 0.50 m from the freezer pipes at all points meant that the thickness of the frozen ground had to be well over 1 m. When freezing was performed, the actual temperatures reached were measured in the zone concerned by introducing chains of thermometers positioned parallel to the freezer pipes. Mathematical models based on the concepts of heat

transmission theories were used (see figure 9) to produce plane thermometric diagrams which displayed the changes in temperature during construction at the design distance already mentioned (0.50 m. from the freezer pipes).

The action hypothesised was therefore fully able to guarantee the safety of the works and to prevent inflows of water underground.

Finally the station tunnels and the stairway access tunnels were driven full face, using a roadheader, followed by the placing of a preliminary lining, water proofing and a final lining in concrete with the ring closed with a tunnel invert (see photo 7). It is not until that stage is complete, and that is after adequate confinement of all the walls of the excavation, that ground freezing stops.

4. STATE OF PROGRESS OF THE WORKS

4.1 Shafts

The Garibaldi, Università, Municipio and Toledo shafts have been sunk so far.

4.2 Station tunnels

- Garibaldi and Università stations: all four of the platform tunnels have been driven;
- Municipio Station: three of the four platform tunnels have been driven. The tunnel on the Toledo station side on the even track is currently at the freezing stage.
- Toledo Station: all four of the platform tunnels remain to be driven from the overhead chamber that has already been completed.

4.3 Stairway access tunnels

- Garibaldi Station: both the stairway tunnels on the even track have been completed;
- Università Station: both the stairway tunnels on the even track have been completed;
- Municipio Station: both the stairway tunnels on the odd track have been completed;
- Toledo Station: both the stairway tunnels on the even track have been completed.

4.4 Line tunnels

The two line tunnels started from the Brin shaft (figure 1) have reached chainage 3+930 (odd track side – Municipio-Toledo section) and chainage 1+500 (even track side – Garibaldi-Duomo section).

4.5 Other works

A shaft is being sunk at Piazza VII Settembre to treat and close any cavities there may be between that square and Dante Station. That shaft will be used for the extraction of both the TBMs without any interference with Dante Station which is already in service.

REFERENCES

Lunardi, P., 2000. "Design & constructing tunnels – ADECO-RS approach". Tunnels & Tunnelling International, Special supplement, May 2000

NAPLES UNDERGROUND

Main characteristics of the line tunnels with mechanised excavation (parallel twin bore on the section CDN-Piazza VII Settembre shaft)

Length of the excavation (for each bore):	3,770 m.;
Diameter of excavation:	6.75 m.;
Outer diameter of segment ring:	6.45 m.;
Inner diameter of segment ring:	5.85 m.;
Thickness of segments:	0.30 m.;
Minimum actual radius of curvature:	180 m.;
Design radius:	150 m.;
Length of segment ring:	1.20 m.;
Number of segments per ring:	6 + key.

Main characteristics of the station platform tunnels with conventional excavation methods (4 tunnels for each station)

Total length of the excavation:	900 m.;
Diameter of excavation:	11 m.;
Lining thickness:	0.80 m..

Main characteristics of the line tunnels with conventional excavation methods (parallel twin bore from Piazza VII Settembre to Piazza Dante)

Total length of the excavation:	200 m;
Diameter of the excavation:	6.80 m;
Thickness of the lining:	0.60 m..

Main characteristics of the stairway tunnels with conventional excavation methods (4 tunnels for each station)

Total length of the excavation:	600 m.;
Diameter of the excavation:	6.80 m;
Thickness of the lining:	0.60 m..

Client:	City of Naples.
General contractor:	Metropolitana di Napoli S.p.A..
Design and project management:	Metropolitana Milanese S.p.A..
Design of the underground works:	Rocksoil S.p.A. (Milan).

Start of works:	2002.
Completion of the works scheduled for:	2011.