Industrialisation of the construction process for the Solbiate Olona tunnel by using "jet-grouting" technology

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ABSTRACT: The Solbiate Olona tunnel represents a very significant project, solving the problem of interference between the "Pedemontana Lombarda" Motorway alignment and one part of the Solbiate built area, overlooking the Olona Valley. After a long stretch of cut & cover tunnel, the tunnel passes underground through gravelly-sandy ground, beneath various residential buildings and existing structures with overburdens of around ten meters. The report describes the advance methods employed, developed according to the ADECO-RS approach, which made complete industrialisation of the construction process possible. A detailed examination of the "jet-grouting" technology employed in the ground is given. This was of great importance both in the construction of the tunnel, using sub-horizontal ground treatment, and for the construction of the portal on the Olona Valley side, where a wall to confine the slope was realized using sub-vertical ground treatment.

1. INTRODUCTION

The new "Pedemontana Lombarda" Motorway – 87 km long and adding almost 70 km of new viability – is a key system for the mobility of its territory. It will be able to move 350,000 vehicles each day through an area inhabited by almost 4 million people. The first step towards this important transportation network is "Section A", the works of which were contracted to the General Contractor "Pedelombarda" S.c.p.A, built by Impregilo S.p.A, Astaldi S.p.A., Impresa Pizzarotti & C. S.p.A. and A.C.I. S.c.p.A. These works began in February 2010 and were completed last December. This first section (14.5 km long) links the existing A8 "Milan-Varese" Motorway and the A9 "Milan-Chiasso" motorway (Figure 1) within a partially urbanized territory, including existing infrastructures such as roadways and tracks, as well as constructions and watercourses. At the junction between Cassano Magnano and Valle Olona, the route alternates trench sections with tunnels, limiting the impact on the territory as much as possible. After under-passing the SP (Provincial Road) 20 and the SP 2 with cut & cover tunnels, the route (as it gets closer to Valle Olona) interferes with part of the inhabited area of Solbiate, which is therefore under-passed in conventional tunneling (Figure 2). It was decided to deepen a first section of the road with a cut&cover tunnel, followed by a conventional tunnel

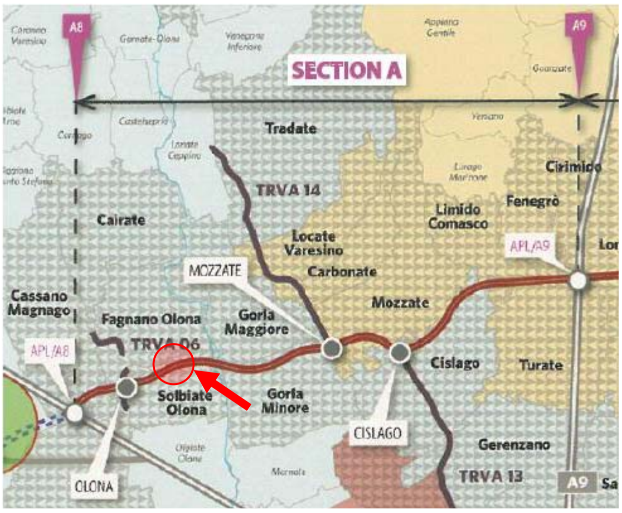

Figure 1. Motorway "Section A".

The tunnel exits on the slope overlooking the right bank of the Olona river; at the end of a cut & cover section, the road meets a shoulder of the

Olona viaduct which crosses the river and allows the route to continue on towards Gorla Maggiore – again in artificial tunnel so as to limit the environmental impact and to preserve the existing local viabilities, such as the SP19.

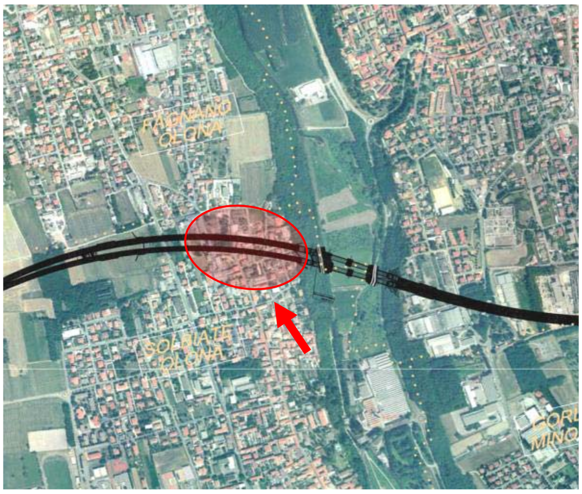

Figure 2. Solbiate Olona tunnel location.

The design of the “Solbiate Olona” tunnel was entrusted to Rocksoil S.p.A., and the choice of the construction methods followed the need to guarantee idoneous safety conditions (both in the tunnel itself, and to the pre-existing structures) and to reach a high level of industrialization of the construction process, to maintain the estimated times and costs. These project choices are presented in detail below, alongside the evidence collected during the execution of the works.

2 SHORT OVERVIEW OF THE WORK

The Solbiate Olona tunnel is made up of two bores (one for each direction), respectively 1,175 m and 1,178 m long; 464 m and 457 m of which in conventional tunnel, with a 35 m distance between the axes. The carriageway is made up of three lanes for each direction; two of them are for traffic (each 3.75 m wide), and the third for emergency (3.00 m wide, with a side platform on the left 0.70 m wide). In order to guarantee visibility on bends for the design speed of 80-140 km/h (in presence of a planimetric radius of curvature equal to 1,350 m and of transversal inclinations equal to up to 5.8%), the east-direction carriageway presents a widening equal to 0.50 m on the right side, while the west direction carriageway presents a widening equal to 2.60 m on the left side. This brings about significant cross sections, with an excavation area of 190 m² for the East bore and 230 m² for the West bore (Figure 3). The two bores are connected with a vehicular bypass and a pedestrian bypass; to these are added also, in the cut & cover tunnel: an additional pedestrian bypass, a pair of emergency refuges and a lateral shaft equipped with emergency staircases, thus following safety prescriptions which require pedestrian connections every 300 m, vehicular connections every 900 m and emergency refuges every 600 m.

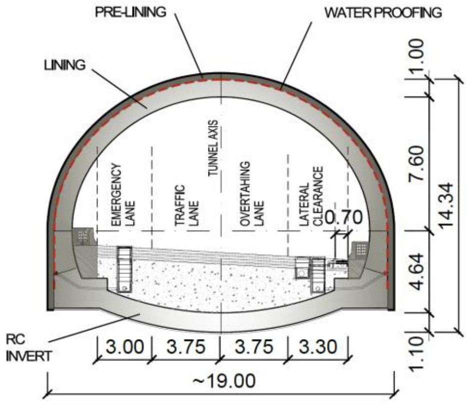

Figure 3. Tunnel section layout.

The tunnel presents very modest crown overburden, averagely between 10 and 13 m, inferior to the excavation diameters of 17 m for the east bore and 19 m for the west bore (H/D ratios within the range of 0.5-0.7). This situation is complicated by the presence of buildings on the ground level for a large portion of the alignment, as can be seen in Figure 4. For the most part these are 2-3 stories buildings built in the 1960s and 1970s, with a mix of load-bearing walls and hollow-core slabs; a few of these are also one-story industrial structures.

Figure 4. Tunnel plan vs pre-existing structures.

In some cases the basement level is 2-3 m below the ground level, as can be seen from the evaluations made to acertain the geometries and the conservation state of the existing structures. The presence of buildings, and the need to keep them inhabited during the works, made it necessary to limit the amount of ground volume loss during the excavation phase, so as to operate control over the subsidence at the ground level and any effects on the pre-existing structures, according to the principles of the A.DE.CO-RS design and contruction approach (Lunardi, 2008).

3 GEOLOGICAL AND GEOTECHNICAL CONTEXT

The tunnel is located within the pleistocenic glacial sediments in the amphitheatres of Verbano and Lario, and the corresponding fluvial and glacial sediments. The fluvial sediments, besides the recent ones, which occupy the current waterbeds, are limited to conglomerates generally called "*Ceppo*", which deposited during the long interglacial period between the late and middle Pleistocene. Following the principle of allo-formations, the "*Allogroup of Besnate*" was found along the tunnel's path. This allo-group belongs to the Verbano basin, and is formed by stratified and graded gravels and sands; Figure 5 reports the geological and geomechanical profile of the tunnel.

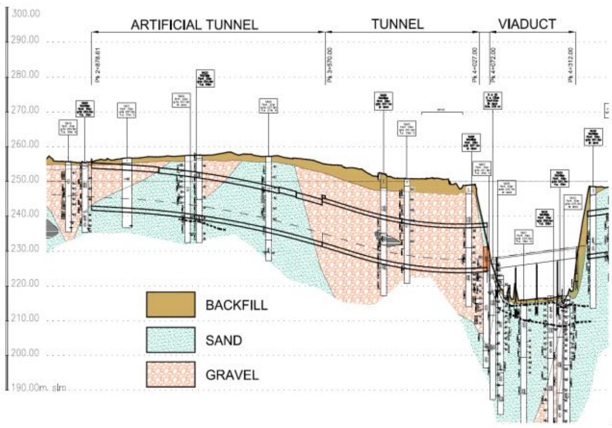

Figure 5. Tunnel geological longitudinal section (overburden 2-13 m, Ø = 19 m).

The area affected by the tunnel also shows surfacing of the "Olona Ceppo", which is made up by clastic support conglomerates with a variable degrees of cementation - ranging from good to bad, to completely absent; many passages are recognizable even upon tunnel excavation faces, as sporadic intercalations (Figure 6). From a geomorphological point of view, the examined area is mostly plain; the only relevant element is the Olona river, which has excavated a slightly etched canyon of small size (about 30 m lower than the main level of the plain), with averagely steep fluvial slopes. Significant forms of instability were not found along the tunnel's alignment; the only potentially instable areas are the fluvial slopes of the Olona river, where superficial strata often give way under bed weather. The tunnel does not present hydro-geological criticalities, as it doesn't interfere with the watertable, the average depth of which is above 35 m, as it joins up with the Olona river watercourse.

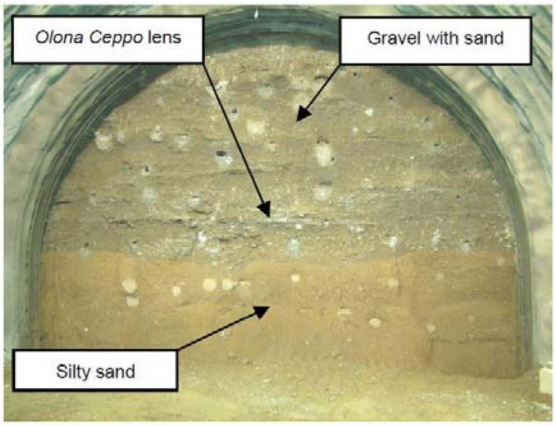

Figure 6. Geological face mapping.

Five geognostic boreholes were used to reconstruct the stratigraphy, while in situ and laboratory tests were necessary for the geotechnical characterization; these tests allowed the grounds affected by the tunnel to be classified as belonging to the unit of "prevalent gravels", whilst the units of prevalent "sands" and "silts" are only present for limited layers. Table 1 reports the parameters of strength and deformation, in function of the depth, used in the numeric analyses.

Table 1. Geotechnical properties of natural soils

Stratigraphy	Depth m	γ kN/m^3	c' kPa	ϕ' kPa	E MPa
Backfill	0-3	18,0-19,0	0	28-30	10-20
Sandy gravel	3-15	19,5-20,5	0-5	32-36	20-50
Sandy gravel	> 15	19,5-20,5	0-5	32-36	50-100
Sandy silty	lents	18,5-19,5	10-15	25-28	10-20

Finally, regarding the seismic site conditions: the peak bedrock horizontal accelerations a_g are equal to 0.054g as regards the Life-Safety Limit State, and 0.031g as regards the Serviceability Limit State; taking into consideration a nominal life V_n equal to 100 years with a class of use coefficient c_u = 2.0 (period of reference Vr = 200 years). For provisory opening works, considering the V_n equal to 10 years (Vr = 35 years), the a_g will be equal to 0.035g as regards the Life-Safety Limit State and 0.016g as regards the Serviceability Limit State.

4 "JET-GROUTING" TECHNOLOGY.

In order to control the deformation response of the ground caused by the excavation, it was necessary to adopt precautionary measures to consolidate the granular material affected by the tunnel, thus improving their strength properties and actively contributing to the stability of the face and of the cavity. The granulometric and permeability properties of the materials made it clear that "jet-grouting" was the best technology for treating significant volumes in short periods of work, especially when compared with other injection and strengthening systems.

Jet-grouting consists of injecting fluid mixtures at an extremely high velocity through one or more nozzles placed at the end of a hollow rod; the fluid jets mechanically break up and permeate the ground, thus cementing it around the injection point. This brings about the formation of columns, the diameters of which depend on the executive operational parameters of "jet-grouting" and on the parameters used (Lunardi et al. 1986; Croce, Gaio et al. 1994). When using one grout mixture only, made of water and cement ("mono-fluid" procedure) the columns formed in the examined granular material are 600-1000 mm in diameter; higher diameters (up to 1,500-1,800 mm) can be achieved by adding compressed air to the cement grout ("bi-fluid" procedure). Due to their potential application both in the tunnel and on the ground level, both procedures were investigated through specific field tests in order to evaluate the idoneity of the technology itself, and to set the optimal operative parameters. This is also in light of the recent evolution that the technology has known in terms of equipment and of the potential of the injection pumps, which are today capable of managing higher injection sections at important pressure levels (400-420 bars). The field tests were carried out during the open-air excavations for the construction of the portal, and they required column treatments with different sets of operative parameters. The columns were then controlled visually for the first 5-6 m from the ground level (Figure 7), until the continuity of the treatment in depth was verified with bore probing.

Figure 7. "Jet-grouting" field tests (1500 mm).

Finally, samples of the treated ground were taken and subjected to mono-axial compressive stress tests in order to define the resistance properties. The results of the field tests were highly satisfying, both in terms of diameter (the bi-fluid procedure brought about diameters within the range of 1,450-1,700 mm), and in terms of continuity of the treatment in depth, where the probes registered RQD > 70 - 80%. The stress tests showed an average compressive resistance level equal to 8.7 MPa, higher than the prescribed level in the project (5 MPa).

Table 2. "Jet-grouting" operative parameters

Procedure	D m	Pr bar	N, mm	Vex cm/min	Vr g/min
Monofluid	600	360-400	3, 2.6	60-80	15
Bifluid	1500	400-420	2, 4.0	30-32	9.38

Table 2 reports the identified operative parameters used during the works; the injection grout had an A/C ratio equal to 1.0. The "bi-fluid" grout used an air pressure of 8 bars with range 8 m^3/min. During the treatment, cement was consumed at a rate of 210-230 kg/ml for treatments 600 mm in diameter, and 800-850 kg/ml for treatments 1,500 mm in diameter. Of great importance is the control over the energy balance with respect to the operative parameters, according to Tornaghi (1993): treatments 600 mm in diameter required specific energies of 11-14 MJ/ml, while treatments 1,500 mm in diameter required specific energies of 42-46 MJ/ml. Figure 8 reports the success of sub-horizontal treatments at the face and around the excavation profile. The company, SGF –

INC S.p.A., carried out all jet-grouting treatments.

5 TUNNEL CONSTRUCTION

Once the best consolidation technique had been chosen, the geometries, the stabilizing actions of the cavity, and the succession of construction phases to adopt were also chosen. As regards the geometries of the treatments (due to the modest geotechnical properties of the ground), pre-containing measures were chosen both for the perimeter of the cavity, and for the core face.

Figure 8. “Mono-fluid Jet-grouting” (600 mm).

Strengthening measures were carried out while advancing to form an artificial “arch effect” upstream the excavation face, with a structural and protective function. Indeed, the deviation of the produced stress, beyond the face, by the shell of consolidated ground both lightened and protected the core face, and limited the detensioning of the ground following the excavation. On one hand, the strengthening of the face guarantees the stability of the core and avoids the release of ground; on the other hand, it minimizes extrusion deformation phenomena and the consequent loosening of ground. Thanks to the two adopted actions, the excavation can take place within stable material, which reflects into a great reduction (from a static point of view) of those actions, which stress the first-phase and final lining, and therefore the deformation behaviour of the cavity in terms of subsidence and convergency. It is therefore possible to operate the excavation “full face”, also in the presence of geometries equal to 230 m², thus favouring the fast stabilization of the cavity thanks to the invert casting.

The availability of space allowed operations to take place from the ground level for the first section starting from the Cassagno Magnano side portal; this allowed the formation of column treatments, which were 1,500 mm in diameter. This also formed a belt of strengthened ground along the perimeter of the cavity equal to 4 m for the crown and 3.5 on the sidewalls, as can be seen in Figure 9 (section type C1c). The geometry of the treatment was studied in order to obtain a squared mesh, downhole, of 1.10 m x 1.30 m, and therefore guarantee the compenetration of the treatments. Drillings were carried out for a length of 19-20 m, with an un-injected and empty section of about 8 m. The excavation face was treated using consolidated diaphragm 4 m thick for every 15m, capable of partitioning the tunnel excavation. The advancement phases were sped up by early use of the treatments (contemporaneously with the rods of the portal retaining walls); for this reason, within the tunnel, solely the excavation and lining phases were made necessary. The pre-lining was put down by using ribs 2IPN180/200 (east/west bore) at the pitch of 1.0 m and embedded within a layer of 30 cm thick fibre-reinforced shotcrete, with 5 cm of regularization. The casting distance of the invert is about 40-50 m from the face. The treatments were carried out in advancement for the rest of the tunnel, following a sub-horizontal truncated-cone shape and nominal diameters of 600 mm.

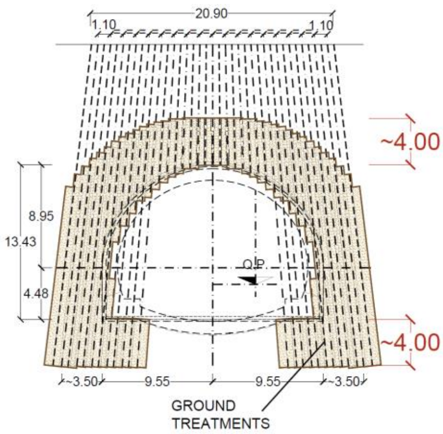

Figure 9. Section type C1c .

The geometries of the operations and especially the lengths of the drillings were studied according to the presence or lack of pre-existing structures on the surface. The section presenting buildings on the surface required double rings of treatment at the outline of the cavity, thus laying a thicker consolidated strip (equal to 1.10 - 1.30 m) and especially, guaranteeing the continuity of the treatment in the presence of local treatment defects. In this case (section-type C1a, Figure 10) treatment lengths equal to 18 m were adopted, with advance fields equal to 8 m; the repetition of the action causes a double overlap along the perimeter of the tunnel with an extra margin of 2 m. The treatments were set at an interaxis of 0.50 cm, with 69 columns in the east bore and 75 columns in the west bore. Consolidation of the core face required the creation of 30 columns in the east bore and 41 columns in the west bore, these are 18 m long and overlapping 10 m (with 4 m of uninjected drilling).

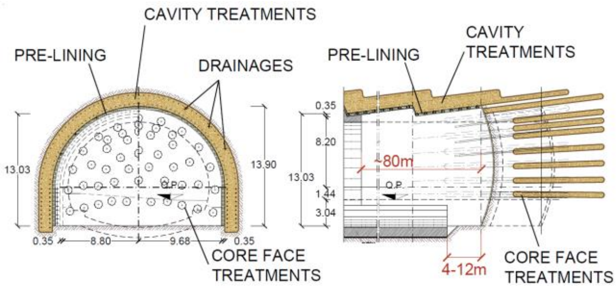

Figure 10. Section type C1a.

The pre-lining is the same of the section-type C1c. The casting of the invert was kept near to the excavation face (varying between 4 m and 12 m – beginning and end excavation of the advance field) in order to minimize the deformation response of the pre-lining. The maximum distance from the crown casting face is equal to 80 m; indeed, the rule is to line the most advanced bore previous to the arrival of the excavation face of the second bore – this has determined casting distances equal to 50 m. Due to the great lengths of the jet-grouting treatments (18 m long), these were controlled during the course of the work, implementing radial probing at the treatment's end. These revealed suitable treated ground thicknesses equal to 140-150 cm.

Where buildings were absent on the surface, the strengthening lengths along the perimeter were equal to 14 m, with advance fields of 9.0 m, and overlapping of 5.0 m (uninjected drilling equal to 1.5 m). In this case, as well, an interaxis of 0.5 m was used, with 69 columns in the east bore and 75 columns in the west bore. The strengthening of the core face used 30 columns in the east bore and 41 columns in the west bore – 16 m long, an overlapping of 7 m (section-type C1b). The pre-linings used were the same as those of the section-type C1a, with the distance of the casting of the invert from the face equal to 15-20 m. The planimetric diagram shown in Figure 4 indicates the sections in which the various section types were used: C1a, C1b and C1c. All final linings, invert and crown, are in reinforced concrete class C25/30 (XC2).

On site, the work cycle was organized into three main phases: 1) treatment, at the face and at the perimeter, 2) advance excavation, for single excavation steps of 1.0 m, followed by the setting of the pre-lining, and 3) the casting of the invert, as shown in Figure 11. Other actions - such as the waterproofing, the setting of the crown reinforcement, and the casting of the final lining – are carried out under the shadow of the critical path, solely conditioning the advancement of the nearby bore, which, due to the reduced septum between the two bores, must flank an already-lined tunnel.

a)
b)
c)

Figure 11. Main execution phases:
a) grouting treatment, b) excavation, c) invert casting.

The average duration of a work cycle was about 7-8 days, distributed as can be seen in Figure 12: 45% of the time was dedicated to the treatments; 40% to the excavation phase, and the setting of the pre-lining, 10% for the invert (Figure 12); a minor percentage was dedicated to collateral activities. The excavation started in October 2011 at the portal on the Cassano Magnano side, and proceeded without interruption and with high regularity.

The construction process resulted heavily industrialized, as can be seen in Figure 13, which exemplifies the production of the east bore in m/month and m/day. Notice the regular monthly production, equal to 30-34 m/month (excluding the months of August and December, with suspensions due to the summer and Christmas holidays), and average daily production of 1.00-1.20 m/day.

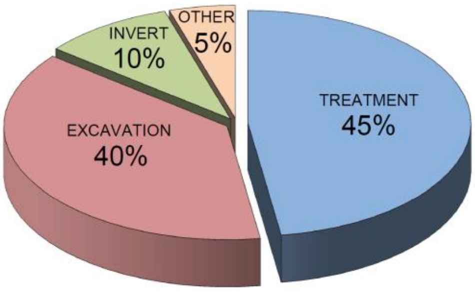

Figure 12. Section type C1b: work cycle .

The advancement (the blue line) is practically linear and a straight line at the start of the work. This stands as testimony to how a careful initial design can help industrialize excavation, and therefore optimize construction times and costs. The drilling of the east bore took place in February 2013, the west bore was completed in April 2013.

6 PORTALS ADOPTING "JET-GROUTING" TREATMENTS

The portal overlooking the valley of the Olona river presents a delicate context, as its morphology is very steep despite moderately resistant granulate grounds. This is due to the local conglomeratic levels ("C*eppo*"), and mainly to a solely apparent cohesion of the grounds, indeed parts of the slopes give way under weather when not held together by the vegetation. Furthermore, immediately behind the slope, residential buildings are present and facing the valley. During the Final Design, in order to prepare the excavation face of the tunnel, a wall of large diameter piles was chosen, to be contrasted by tie-rods during the lowering phase. However, it was during the yard preparation phase that logistical problems cropped up regarding the transportation of equipment for the production of the rods.

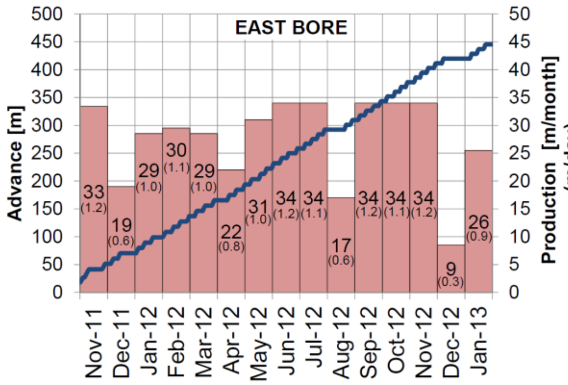

Figure 13. Production (east bore).

Moreover, the foreseen phases for the construction of the portal (which included a series of steps where the excavation would be lowered, alternated by the construction of contrast levels with tie-rods) would have taken up an excessive amount of time. It was therefore decided to use a wall made out of penetrated jet-grouting columns as a way of containing the excavation. This technique has often been used from the 1980's onwards as a way of containing difficult terrains (Lunardi 2008); it was often applied during the construction of the Rome-Naples, and Bologna-Florence stretches of the High Speed/Capacity rail line. The jet-grouting shells create a pre-containment structure on the ground upstream the future excavation, preserving the pre-existing natural equilibrium thanks to the formation of an "arch effect" capable of minimizing movements on the slope. Furthermore, jet-grouting portal wall allow conventional excavation with minimal overburden (1-3 m), thus reducing the height of the wall and the pre-earthwork operations on the slope.

Figure 14 shows the geometries of the action; due to the presence of two bores, a wall was constructed with two arches and a central pillar (constructed with jet-grouting columns). The height of the excavation varies along the perimeter of the wall, from a few metres at the side wings to a maximum of 16 m at the west bore axis. The wall is constructed with three jet grouting column treatments at the greatest heights ("bi-fluid" grout, nominal diameter 1,500 mm, inclination 5°, 7° and 15°), creating a septum both planimetrically curved and slanted in height, and therefore capable of contrasting the pressure of the ground. The treatments are set on the top at a diagonal interaxis of 0.50 m and a longitudinal axis of 1.0 m; the middle line is not reinforced and its first section is uninjected drilled.

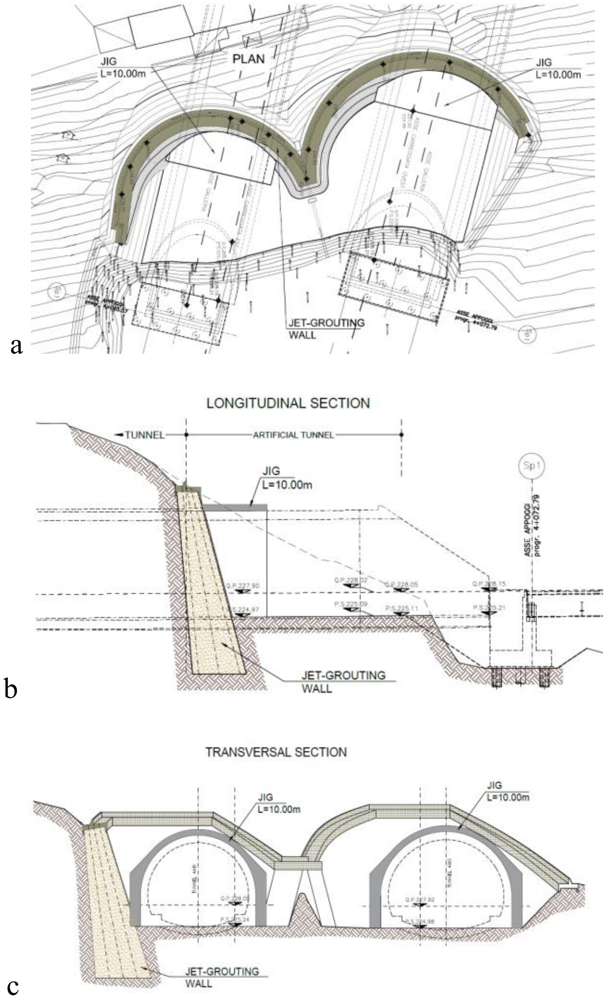

Figure 14. Portal jet-grouting wall: a) plan, b) longitudinal section, c) transversal section.

Two columns were prepared for excavation heights below 10-11m, these columns are 7.5° and 15° inclined. The lengths of the treatments vary from 5-8 m to 23 m, with penetration lengths between 4 and 7 m; the external columns are strengthened with steel pipes (diameter 88.9, width 10 mm), positioned within the treatments following re-drilling. The column treatments are "capped" at the top with a reinforced-concrete beam, which stiffens along the axis of the wall and guarantees the monolithic nature of the operation. Around ten passive tie-rods were placed at the highest sectors as a precautionary measure at an interaxis of 4 m; barbicans were also set locally in order to avoid hydrostatic pressure following intense weather. The portal was prepared following 2-3 lowering phases, a protective layer in strengthened shotcrete with an electrically welded mesh was set at the end of each of these phases.

The opening works (portal) were verified using two-dimensional FEM analyses, to verify the statics of the supporting work, as well as the changement in the stress-strain profile of the slope. Careful attention was placed on the planimetric movements and subsidence of the buildings behind the portal. The analysis followed a series of phases: after reconstructing the geostatic state and after simulating the loads transmitted from the foundations of the constructions, the construction of the wall and the following lowering phases were also simulated. The analyses reported horizontal movements inferior to cm (Figure 15), as shown also from the topographic monitoring; movements below the foundation levels were equal to a few mm.

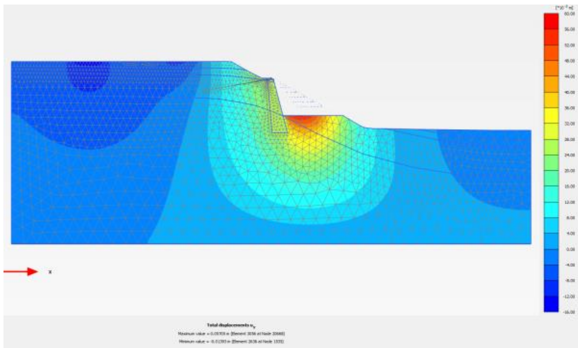

Figure 15. Numerical analysis: horizontal movements.

Static evaluation of the wall was carried out by taking care that the ratio between mobilized τ and limit τ - derived from the breaking criteria of the jet-grouting material – remain constantly below the unit (the ratios remained within the ranges 0.23-0.59); the traction stresses were also verified – partially absorbed by the metallic tubes – as well as the maximum compression values (0.81 MPa, the project value was 5 MPa). Finally, preliminary three-dimensional analyses were also carried out, in order to verify the effect of the curvature on the containment of the stress acting upon the supporting work.

The construction phases for the preparation of the portal are: 1) creation of the wall with jet-grouting treatments, 2) lowering excavation while working on the necessary sub-horizontal treatments for the underground opening treatments, 3) construction of the support jig; the main phases are illustrated in Figures 16. The works began in July 2012, with the preparation of the work plane for the treatment actions. Due to the lesser encumbrance of the machine used (in comparison with the equipment necessary for the creation of large diameter piles) access to the work areas are less invasive, in some cases stabilized material was brought back in order to avoid incising the slope. The treatments on the east bore were carried out in less than 3 months, including the construction of the end beam.

Figure 16. The executive phases: a) working area, b) treatment execution, c) core face treatment, d) jig construction, e) cast-in place portal.

The lowering of excavations was carried out afterwards and took only a few months (without tie-rods), including the grouting in advancement for the tunnel excavation. Following this solution rather than using piles saved a few months' work and led to a more stable slope, as well as competitive costs.

7 MONITORING EVIDENCE

Control of the deformation response caused by a 230 m² excavation section was carried out by means of a monitoring plan requiring systematic cavity convergence measurement stations within the tunnel; these used optic targets placed on the pre-lining (1/advance field). A topographic levelling network was also installed at the ground level. Furthermore, optic targets were used to measure the extrusive behaviour of the face during long pauses due to festivities and holidays, as well as main stress-state measurement stations which were kept in place during the first part of the work. The collected data confirmed the deformation response that had been foreseen by the numeric analyses during the project phase: the core face proved to be "stable" after the treatment, with a few negligible deformations; during long advance pauses the face-movement measures indicated extrusions below the cm (proof of a deformation response in elastic domain). The cavity movement measures also gave very small convergence values, in the 3-8 mm range, below the defined range limit of 25-50 mm (convergences above 10 mm – up to 21 mm – was registered in only 3 stations out of 86). The most significant monitoring regarded the measure of the subsidence at the ground level, due to the possible effect on the pre-existing structures. The topographic measuring systems indicated subsidence in the range of 15-25 mm, both at the ground level and on the buildings, in accordance with the project forecasts, and which never went beyond the attention and alarm levels (35 mm and 50 mm, respectively).

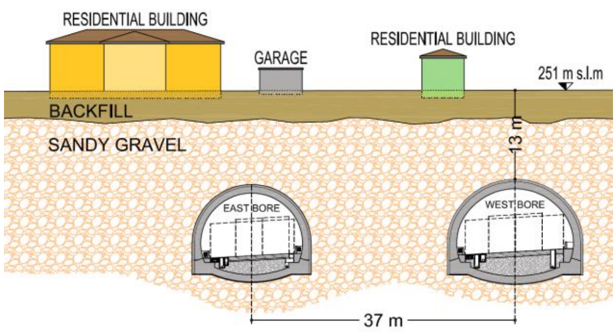

Figure 17. "Via Opificio Valle" cross-section.

The measured subsidence proved that the foreseen consolidation actions and the succession of executive phases – especially the use of the invert on the face – brought about very low levels of volume loss. As an example, the subsidence basin found at the ground level near the diagonal section of Via Opificio Valle seen in Figure 17 (indicated "section A-A" in the planimetric chart of Figure 4). Subsidence was measured on an alignment diagonal to the tunnel axis, remaining within a range of 2-24 mm and following a Gaussian progression (blue line). The amount of "lost value" under the monitored subsidence curve was equal to 0.3% of the excavated volume – this was much less than that generally reported by the literature for traditional excavation (where the "lost volume" is generally around 1.0-2.0%). Always according to Figure 18, the subsidence basin under consideration is reported using the usual empirical formulations (Peck, 1969, O'Really and New, 1982) with a volume of "lost volume" equal to 1%, despite how the numerical evaluations use analyses of finished differences – with FLAC calculation code – in the absence of actions and using the projected jet-grouting treatments.

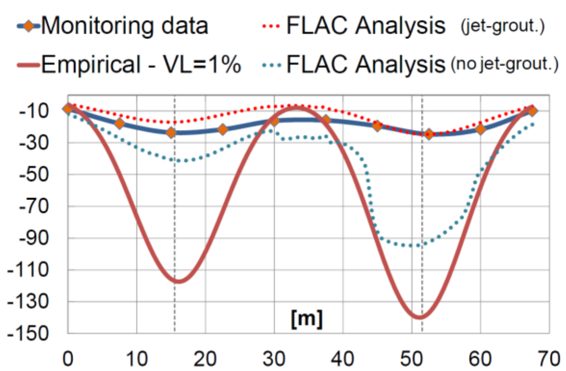

Figure 18. Ground level subsidence (mm).

Notice how this last simulation fits in very well with the monitoring data, proof of how the advancement modalities used were successful in keeping the disturbance on the mass extremely low, in line with the mechanical system excavation solution.

8 FINAL COMMENTS

Experience gathered while constructing the Solbiate Olona natural tunnel proved that "jet-grouting" is an efficient system for the treatment of above-waterbed granulate grounds, such as gravels and well-graded silts. This technology is capable of creating pre-containment structures in the grounds before proceeding towards excavation, preserving the natural balance with the creation of "arch effects" which minimize movement in the materials. This is both in the case of underground excavation (for the construction of conventional tunnels) and in the case of open-air excavation, for the creation of opening works. In the here-present case, use of the technology underground allowed full-face advancement with excavation diameters up to 19 m (A_{exc} = 230 m²) with cover up to 10-13 m and buildings on the surface which remained inhabited during the excavation. Maximum subsidence recorded was in the range of 15-25 mm, with a "loss of volume" equal to 0.3%, in line with mechanical excavations. The excavation methods, which followed the ADECO-RS approach, also allowed a full industrialization of the construction process, with regular productions equal to 30-34 m/month (or 1.0-1.2 m/day). The "jet-grouting" technology was successfully adopted for the construction of the tunnel portal in the delicate Valle Olona slope. The logistical problem of using the equipment necessary for piles was solved thanks to jet-grouting column treatments, minimizing the deformation impacts on the slope; the solution also helped optimize costs and time (at the same price), thus avoiding an expensive alternating between excavation phases and the placement of contrast tie-rods.

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