Chapter 4

Auxiliary methods technology

Ground reinforcing, ground improving and pre-support technology

G. Cassani, M. Gatti, and C.L. Zenti
Rocksoil S.p.A.

V. Manassero
Underground Consulting S.a.s.

S. Pelizza
Politecnico di Torino

A. Pigorini
Italferr S.p.A.

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4.1 INTRODUCTION

Often soils and rocks need to be reinforced and improved to contribute to tunnel stability and to contribute to controlling tunnel deformation phenomena. Auxiliary methods, also named consolidation works, are therefore very important tools in tunnel design and need to be deeply known with reference to their specific technological characteristics, so as to understand the different application fields according to the soil and rock "virgin" geotechnical characteristics and to the operational context. The most frequently used auxiliary methods have been generally described in Volume 1 (Section 8.4), while in this chapter, these methods will be thoroughly described with the aim of providing a practical help to define the main parameters for the use of the specific intervention (for example, injection pressure, mixture volume, mixture composition and pipes and injection element characteristics) and of defining the achievable results in terms of increased geotechnical properties of the ground (resistance, deformability and permeability) after the interventions. The main technical characteristics about machinery and equipment to be used will be also given together with production rates.

4.1.1 General classification

There are a series of consolidation techniques available to the design engineer to perform different types of ground reinforcing and improving to obtain the appropriate action for a given stress-strain situation, in relation to ground physical characteristics. According to different classifications proposed in the literature (Figure 4.1; Lunardi, 2008), they can be grouped either in relation to the position of the action performed:

- ahead of the tunnel face (preconfinement and pre-support interventions) or behind the tunnel face (confinement interventions);

  or in relation to the action performed:

- the main action may consist in confining the decay of the minor principal stress – σ^3 (conservative), or in increasing the share stress of the ground (improvement).

Performing consolidation works ahead of the tunnel face contributes to anticipating and to better controlling ground deformations (pre-convergence and extrusion) before it develops with tunnel face progression (convergence): those interventions allow both to protect and to reinforce the advance core in order to transform the core of the ground ahead of the tunnel face into a "structural element" with characteristics tailored to solve tunnel stability and deformation problems (Figure 4.2).

4.2 STEEL PIPES UMBRELLA AND SPILING (PRE-SUPPORT INTERVENTION)

The "umbrella arch" family counts of several ground reinforcement systems that are applied in unstable ground conditions above the water table defined by a very limited

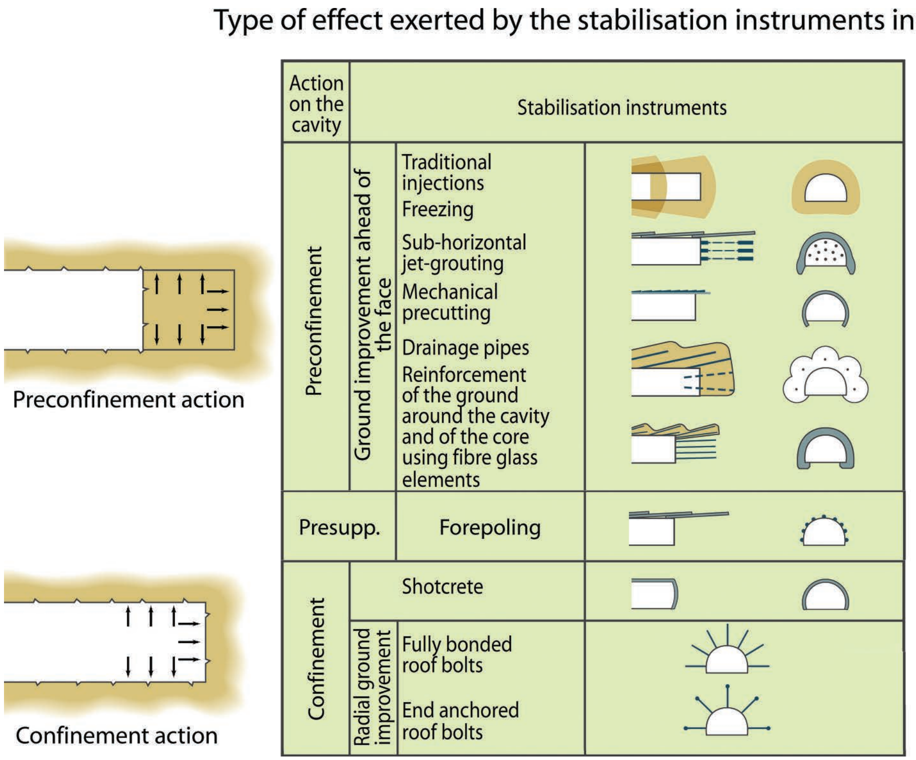


Figure 4.1 Auxiliary methods classification in relation to the position where they are implemented regarding the tunnel face (preconfinement, pre-support and confinement) and to the main effects performed. (Modified from Luna 2008.)

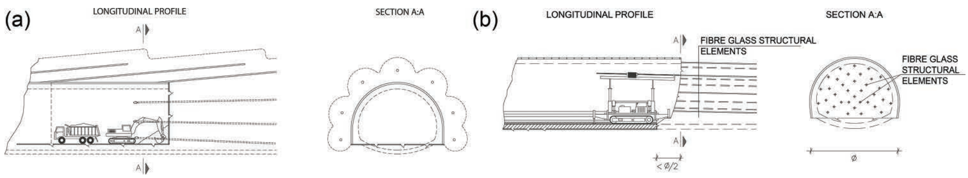


Figure 4.2 Examples of drainage at the contour of the cavity and reinforcement of tunnel core with fibreglass elements, performed ahead of the tunnel face (preconfinement interventions; Lunardi, 2008).

stand-up time. As already discussed in Chapter 8 of Volume 1, the terminology used for these interventions is quite confused and worldwide the same type of intervention can be called in different ways, but the basic concept is the following: these systems are auxiliary methods with the purpose of reinforcing and temporarily supporting the short section of the tunnel upon excavation, improving both the safety of workers and the stability of the unsupported tunnel boundary.

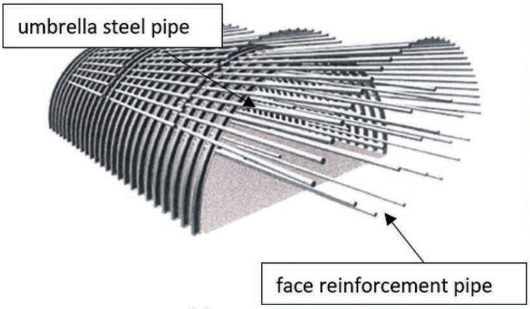


Figure 4.3 (a) Sketch of the steel pipes umbrella arch method. (b) Face reinforcement installation at the face.

In a general scheme, the method consists of inserting steel pipes or other types of long structural elements inside the ground to form a protective structure that is further supported with steel ribs once installed (frequently, when the umbrella arch is applied, also the face requires pre-reinforcement; Figure 4.3) (Okak, 2008; Oke et al., 2014).

4.2.1 Historical overview: the origin of the method

The oldest description of this technique can be found in the book "The Attack and Defence of fortified Places" by Muller (1757), where it is reported: "The gallery being carried on some distance, carpenters follow the miners to prop earth above, to prevent it falling in. This is done by placing a piece of wood across the Gallery overhead and putting a prop or upright at each end, fastened into the ground, then the earth is reminded above this place, so as to slip a piece of deal board over it, which being afterwards supported in the same manner of the other end will sustain the hearth" (Figures 4.4 and 4.5).

The method is known in England as piled heading or poling board, while miners call the method spiling, and in the USA, it is called forepoling (in Italian language "marciavanti" or "infilaggi"; in French language "enfilage"). The principle behind this method is to ensure that not more than the minimum possible of poor ground is exposed and that is quickly covered by some kind of plates as timbers or steel boards or shotcrete.

4.2.2 Modern review of the umbrella system

The system consists of drilling and subsequently grouting a series of long hollow steel tubes along the crown of the tunnel that are installed around the excavation face in an umbrella shape and then grouted to form a protective arch that is further supported with steel sets. The construction of the tunnel is done by subsequent sessions as requested by the continuity of the reinforcement, therefore each section has a truncated cone excavation geometry (Figure 4.6).

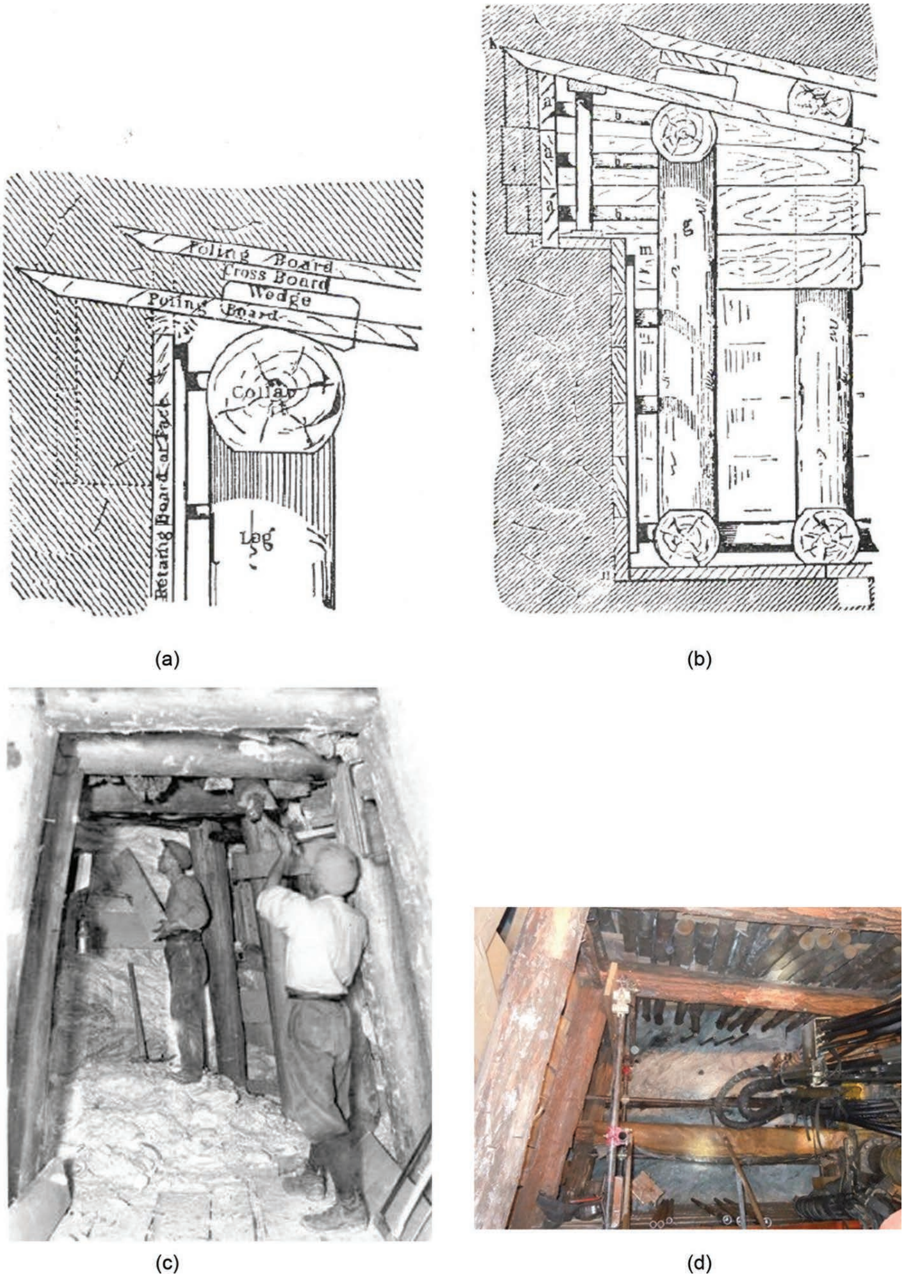


Figure 4.4 Example of pictures of forepoling taken from the lost art of forepoling (Sandstrom, 1963). (a) and (b) show a later phase prior to excavating the ground in front of the square set (b) and replacing the breast-boards (a) with a square set close to the face. (c) and (d) show the application of this technology in a talk mine in Italy to support the excavation steps in drifts in a poor talc rock mass still used.

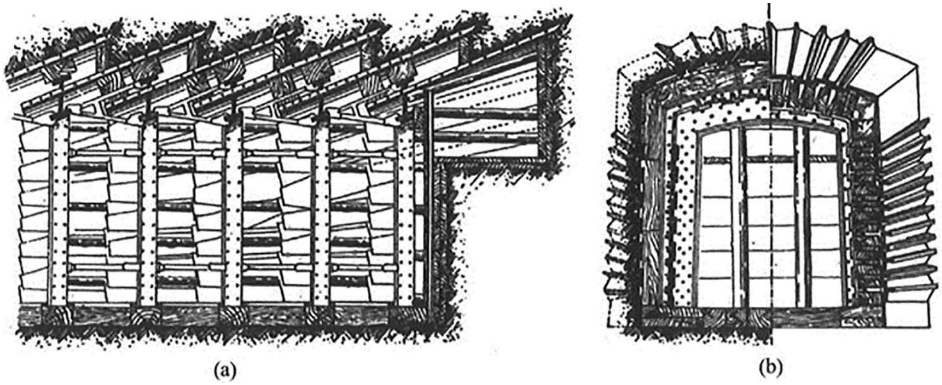


Figure 4.5 Special reinforced forepoling was used to cross a problematic failure in the Tenda railway tunnel (1890–1900). In this particular case the wooded boards were reinforced by iron sheet plates. On the upper part of the face is shown the first new attack with square sets pushed against the face (Callari & Pelizza, 2020).

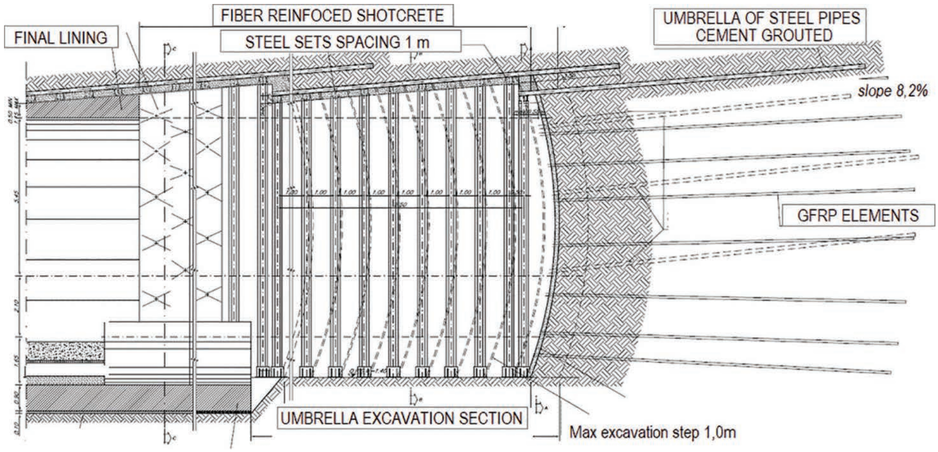


Figure 4.6 Example of steel pipes umbrella arch method with fibreglass face reinforcement.

The need to drill and install long pipes requires the use of a truncated cone excavation geometry to avoid the interference of drilling equipment with the already excavated stretch of the tunnel. Excavation proceeds under the protection of the grouted tubes, by excavating and installing the primary support for cycle lengths in the range of 8–9 m. A 3–4 m pipe overlapping between consecutive umbrella arch rounds is necessary. Under the protection of the umbrella, the excavation is done by subsequent advancing steps long as the spacing between the steel sets. It is important that each advancing step is completed with steel arches and shotcrete before starting the excavation of the next one. The papers of Carrieri et al. (1991, 2002) and of Pelizza (2016a,b), as well as the bibliography and references, provide lists of various case histories (Figure 4.7).

Figure 4.7 Example of a correct installation of the umbrella arch and the face reinforcement and of a correctly executed steel pipes umbrella in a parabolic shape tunnel supported by a parabolic steel rib.

In recent years, advances in perforation and injection techniques have permitted to extend the field of application of the method to fine sands characterized by a very straight granulometric curve, transforming it to a universal excavation method for unstable ground conditions even in the presence of water.

In any case, pipes drilling and grouting operations result in slow advance rates: in general, six to eight working days are required for an 8–9 m cycle of tunnel consolidation and excavation. This is the reason why tunnel personnel sometimes omit steps of the application procedures, resulting in serious consequences for the local stability of the tunnel. Experience shows that lessons are learnt better from mistakes than from a work done correctly. The most frequent mistakes are listed hereafter:

- *absence of the first steel set*: it leads to errors in geometry/positioning of the umbrella pipes as well as in losing the first support of the pipes that works as a weak cantilever (Figure 4.8);
- non-uniform drilling of pipes and/or deviations from the design longitudinal inclination resulting in larger spacing between adjacent tubes and subsequent ground running/flowing or not correct contact with the steel ribs (Figure 4.9);

Figure 4.8 Examples of the failure of the umbrella arch due to the absence of the first rib. This mistake contrary to the design requirements resulting in bending the steel pipes and tunnel collapse.

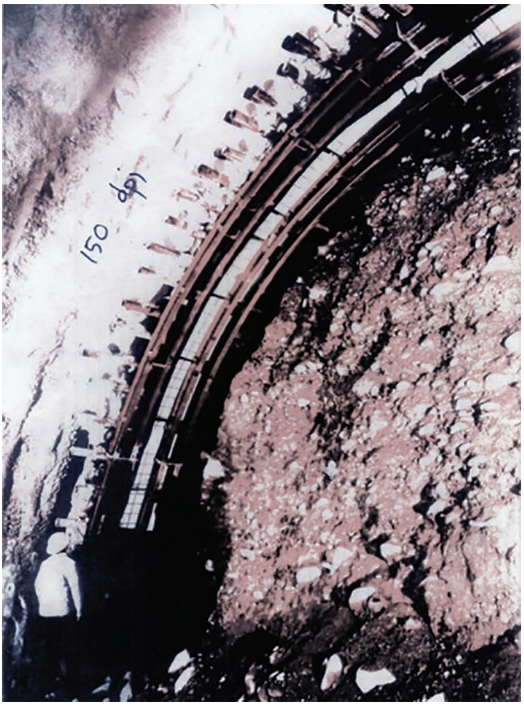


Figure 4.9 Improvement of the contact between the steel pipes umbrella and the supporting ribs. In general, the drilling is not in a correct position because of the deviations, so the steel pipes umbrella does not touch correctly the ribs. Therefore, it can be necessary to install on the extrados of the rib a geotextile bag or similarly that is swollen with mortar regularizing the contact between the pipes and the ribs.

- omission of steel set installation according to the pre-defined design spacing, resulting in a too long excavation length (Figure 4.10), that is a quite common and serious mistake that results in the bending of the steel tubes and face collapsing;
- poorly prepared cement grouts and/or poor grouting of the pipes is also the cause of failures;
- in an attempt to save time, drilling (without grouting) of the entire, or an important sector of umbrella arch is a common and unfortunate practice. The negative consequence is obvious: the ground beyond the face and around the crown is precut by hundreds of metres of drill holes, which causes its weakening and drainage, partly transforming it into a muddy medium;
- inadequate reinforcement of the excavation front, which collapses and/or excessively deforms, resulting in yielding or even collapsing of the umbrella pipes (Figure 4.10).

The calculation approach for steel pipes umbrella can be found in Chapter 10 of Volume 1.

In Figures 4.11–4.14, some examples of failure of steel pipe umbrella or special problems are presented (Girard, 1987).

Figure 4.10 Example of a too long excavation. The excessive length of excavation can determine an excessive load on the end of the excavated span and it has caused the collapse of the face, followed by the collapse of the pipes.

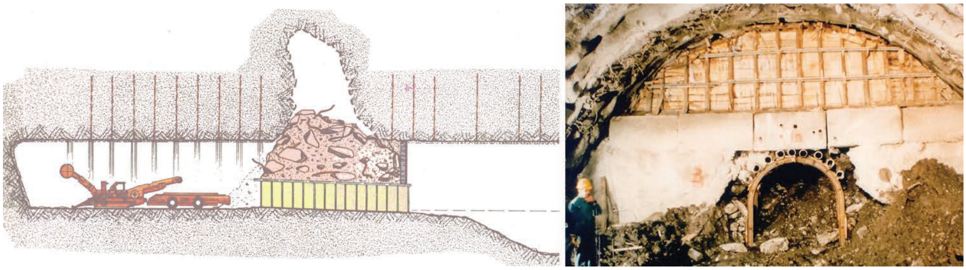


Figure 4.11 A local collapse occurred while benching in a road tunnel in flysch. Because of a working machine is trapped beyond, a consolidation by grouting was impossible. It has been necessary to pass through the heap of loose ground to seal it. This short drift was open under a special steel pipes umbrella arch nearly sub-horizontal.

4.2.2.1 Examples of steel pipes umbrella applications

EXAMPLE 1: INNOVATIVE APPLICATION OF SELF-DRILLING STEEL PIPES

The reconstruction of the Serra dell'Ospedale motorway tunnel on the Salerno-Reggio Calabria motorway (Italy) had to face very adverse geotechnical conditions of loose fine sands alternated with silty clays and clayey silts (Bel Castro et al., 2011). Because of ground conditions, even small-diameter boreholes collapsed upon drilling. A new remarkable adaptation of the steel pipe umbrella combined with a new face reinforcement method was put into practice. According to this new method, both umbrella arch and face reinforcement consist of self-drilling steel pipes injected with cement mortar at

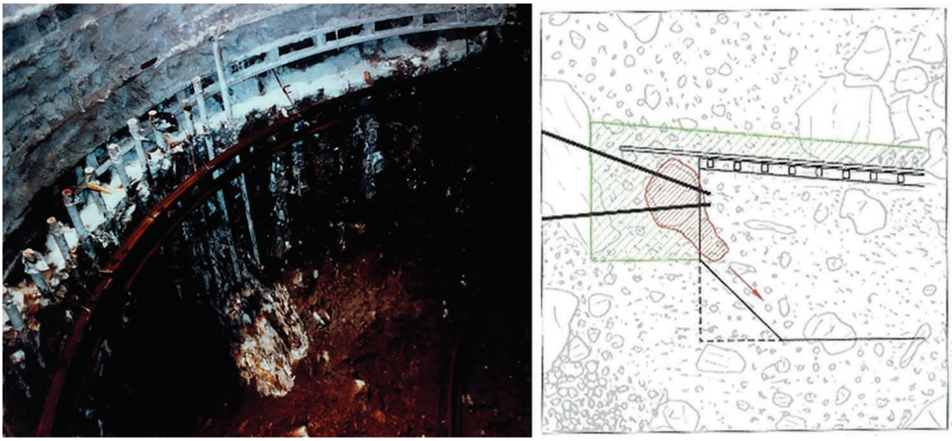


Figure 4.12 The excavation of a glacial morain or slope debris with big blocks has to face the easy instability of the blocks when appear out of the excavation ground surface. Steel pipes of the umbrella on the face are very useful tools to reduce the mobility of the blocks.

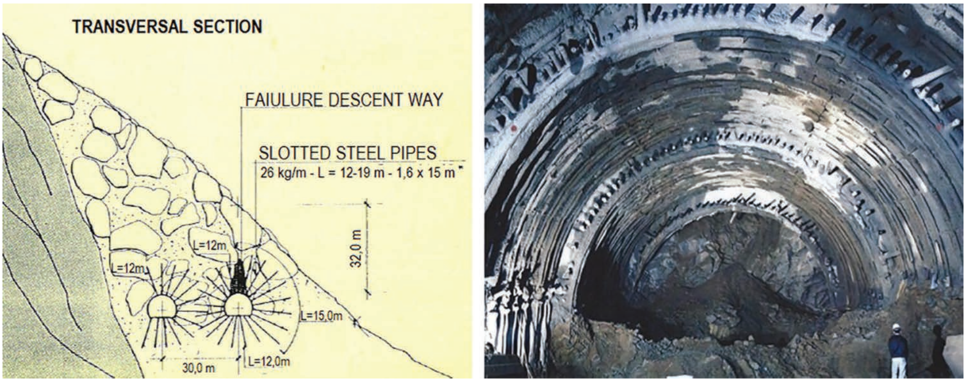


Figure 4.13 A motorway tunnel excavated in a Paleo-landslide with big blocks of hard micaschists with some interposed voids. The ground is pre-reinforced with an umbrella arch of 55 big steel pipes pulled together, but not cemented because of the voids in the ground.

200 bar. This new technique completely eliminated the problem of borehole collapsing. In addition, the application of this method resulted in significant time savings as the operations of drilling, inserting and grouting the pipes were replaced by one single operation. Robust load-bearing pipes with a diameter of about 110 mm and a thickness of 8 mm were used for the umbrella arch, while smaller pipes were used at the face (diameter 60.3 mm and thickness 8 mm) as they were installed for greater lengths (Figure 4.14).

During the excavation, which was carried out with a simple scraper, these pipes were first bent and then cut with special mechanical shear every two or three rounds,

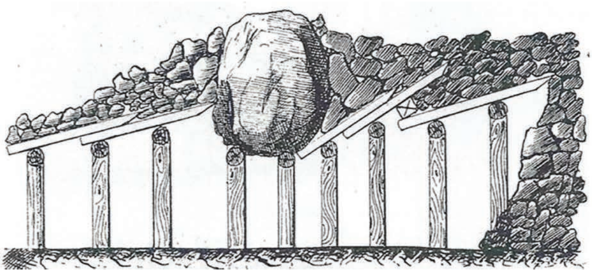


Figure 4.14 Underpass of big boulder in an antique mine (left) and in a modern motorway tunnel (right). The motorway needs to go underground because the boulder is part of a wide Neolithic human settlement that has to be preserved.

leaving the muck uncontaminated, free of usual fibreglass pipes. The conclusions that can be drawn from the use of this new method are briefly summarized hereafter:

- the optimal not injected drilling length is about 2.0 m, which results in a low reflow from the borehole top;
- the consumption of mixture varies from 130 to 220 kg of cement per metre of injected length;
- drilling/injection speed is 8/10 seconds per 12 cm of drilling;
- the injected volume in the sands had a corresponding radius varying from 30 to 45 cm; the adherence between the steel pipe and the treated ground was guaranteed by the self-swelling property of the mixture (Figure 4.8);
- the steel pipes adopted completely eliminated the problems of blocks falling from the face.

Consolidation and excavation of a 6 m long round required eight working days (three shifts per day).

EXAMPLE 2

The tunnel has to overpass the antique Cuma sewer from Naples and Pozzuoli built at the 19th century, that cannot be put out of service.

The tunnels are excavated within mixture slightly coesive of fine and course send of vulcanic and marine origin.

The distance between tunnel and Cuma sewer extrados is only 1 m.

Two kind of steel pipe umbrella pre-support were used (Figure 4.18):

- below the tunnel base: a flat canopy consisting of horizontal piles installed between the future tunnel and the Cuma extrados was drilled from a trench inside of the tunnel using directional drilling;

  this horizontal support structure is found on vertical micropiles;
- the tunnel was pre-supported with a more conventional steel pipes umbrella arch.

4.2.3 Forepoling and spiling

The intervention ahead of the face can also be obtained not with pipes, but with other types of shorter elements usually not longer than the tunnel height or even shorter. In this case, the most used terminology is forepoling or spiling. With this terminology is described an auxiliary method used to stabilize the ground around the crown of the tunnel that cannot have an effective structural capacity by itself because it uses short and not very resistant elements (compared with the previously mentioned umbrella pipes technique) such as rock bolts or self-drilling pipes (not longer than 6m); furthermore, these elements are not in direct contact with the steel ribs. The aim of these auxiliary method is to create a reinforced zone of the fractured rock ahead of the excavation that will guarantee the tunnel boundary stability. The excavation in this case has a constant cylindrical geometry (Figure 4.15).

A very special case of spiling has been used for a particularly complicated segment of Turin metro Line 1 (Alessio et al., 2009). The considered stretch is located in a densely urbanized area, under buildings of high historical value, in the presence of utilities networks some of which could not be interrupted, at a neuralgic point of public transportation, in an unstable granular soil (loose gravel with few fines) that cannot be left unsupported; last but not least, the metro alignment at this sector has a straight 55m radius curve located at the intersection of Corso Vittorio Emanuele II and via Nizza (impossible for a TBM excavation). The excavation area is 50m2 (Figures 4.16–4.20).

The method adopted was to reinforce the crown of the tunnel by forepoling consisting of short self-drilling bolts injected with a bi-component organo-mineral silicate-based grout. The use of this resin guaranteed the formation of a consolidated layer of steel-reinforced ground around the tunnel (with a thickness of about 30cm and strength and compression characteristics varying between 3 and 3.5MPa) in a few minutes allowing for the immediate excavation restart upon completion of the injection. The successful application of the method resulted in zero impact on the city (no traffic and sewer deviations) (Figure 4.21).

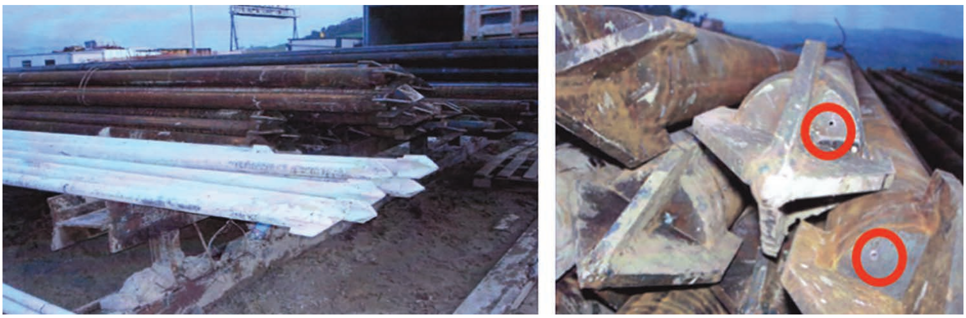


Figure 4.15 Example of the self-drilling pipes with a simple bit used in Serra dell'Ospedale tunnel (the red circles highlight the outlet nozzles under pressure of the mortar for the mini-jetting).

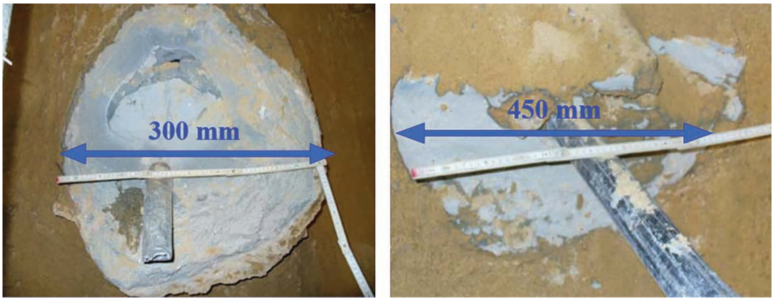


Figure 4.16 Pictures of the obtained columns injected at 200 bar through the 60.3 mm diameter self-drilling pipe at the face: the column is circular with a radius of about 30 cm; the column extends more in the horizontal direction due to the local presence of a layer of loose sand.

Figure 4.17 Detail of scraper-excavating machines, the drilling pipes are first bent by the excavator and then cut off with a robust shears that is driven by the excavator pivoting arm.

Average advance rate was 1 m every 12 working hours. The excavation cycle was defined by:

- Installation of the 37 self-drilling bars: 2 hours.
- Injection of the bi-component resin: 6 hours (with a quantity of resin that varied between 20 and 25 kg per metre of injected bar).
- Full section excavation of 1 m advance in 2.5 hours.
- Installation of the preliminary lining (steel arches and shotcrete): 1.5 hours.

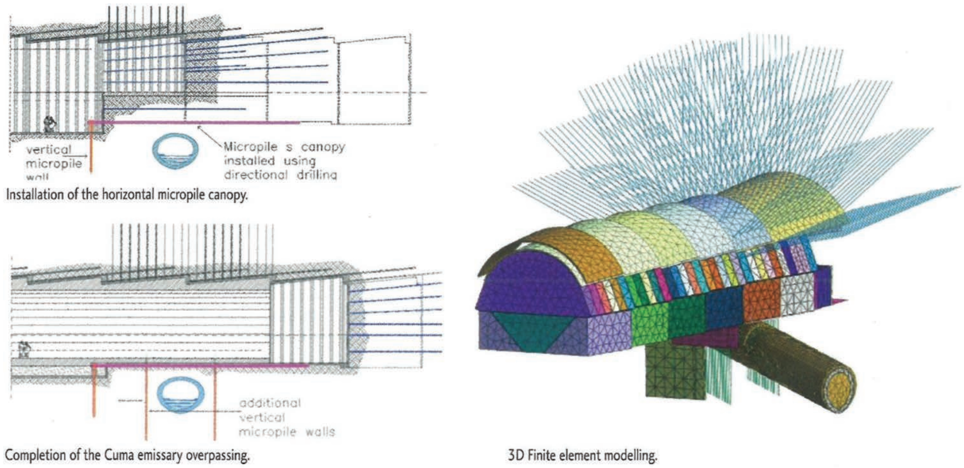
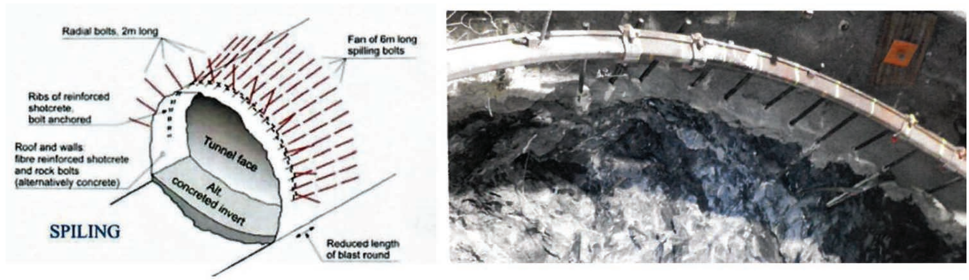


Figure 4.18 Example of the overpass of the antique Cuma sewer from Naples and Pozzuoli.



(a) (b)

Figure 4.19 Example of forepoling with bolts: Spiling: (a) sketch; (b) a spiling local support used with yielding sets (T.H. type) to create some local and temporary stability of pieces of rock, at the same time avoiding future difficulties for the re-enlargement of the tunnel.

4.2.4 Summary

The umbrella arch method includes several ground reinforcement techniques at the excavation face and its boundary. These systems are auxiliary tunnelling techniques that consist of a scheme of grouted steel pipes around the tunnel crown ahead of the face creating a pre-support under which excavation normally proceeds despite the unstable ground conditions. The paragraph describes the origins of the method and presents, through actual cases, innovative variations of the method as well as the similar methods of forepoling and spiling (Figure 4.22).

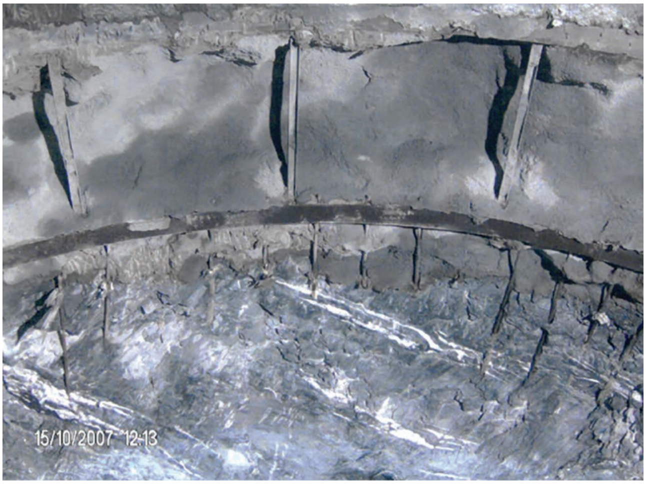


Figure 4.20 Picture of forepoling with steel bars that passing through bores in the steel ribs.

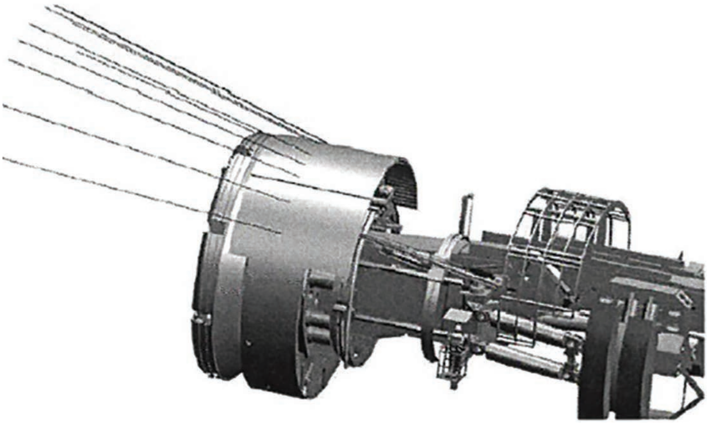


Figure 4.21 The forepoling system can also be used to reinforce the ground immediately ahead of the cutterhead of a TBM.

4.3 USE OF FIBREGLASS ELEMENTS FOR FACE REINFORCEMENT (PRECONFINEMENT INTERVENTION)

Fibreglass reinforcement is one of the most used techniques to improve the strength parameters of the ground around the excavation and/or to stabilize and control the deformation of the face of tunnels driven under difficult ground conditions. It is considered as one of the most economical and effective provisions for ground improvement, first

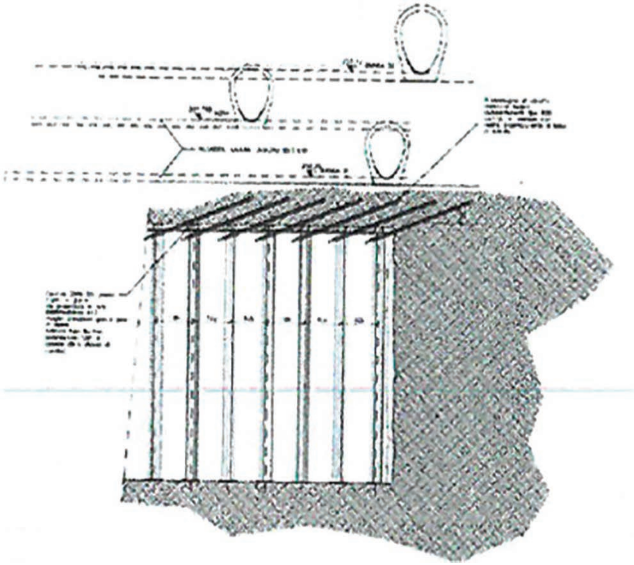


Figure 4.22 Example of the use of forepoling in Turin Metro Line 1. Longitudinal tunnel section.

introduced for the reinforcement of natural and engineered slopes (Schlosser, 1982; Juran & Elias, 1991) and later extended to other engineering applications, such as underground excavations (Mair & Taylor, 1997). In tunnelling works, its use has become integral part of design methods (Figure 4.23), its effectiveness being recognized in the control and reduction of the tunnel pre-convergence and face extrusion (Lunardi, 2008).

In the recent years, efforts have been devoted to reaching a better understanding of the mechanical behaviour of both new materials and equipment adopted for these techniques, which are continuously subjected to technological improvements (Zenti, 2006; Cheng et al., 2009; Kim et al., 2009). At the same time, analytical and computational procedures have been worked out, so that it is nowadays possible to take into account the mechanical reinforcing action of nailing systems at the design stage (e.g. Wong et al., 2000; Ng & Lee, 2002; Anagnostou & Serafeimidis, 2007; Oreste & Dias, 2012).

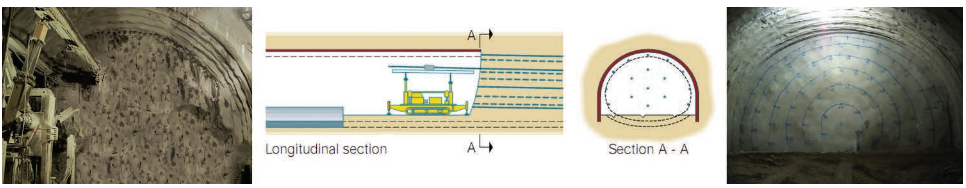


Figure 4.23 Fibreglass elements for face reinforcement (Lunardi, 2008).

The reinforcing elements made of glass fibre-reinforced polymer (GFRP) are adopted since the nineties in all the applications where GFRP's characteristics represent an advantage (Ortigao, 1996). GFRP elements for face reinforcement offer high tensile strength, low unit weight and high resistance to corrosion and can be easily removed by cutting. The latter property is particularly relevant whenever the removal of the already nailing stabilized ground mass has to take place, such as during tunnel face advance. Moreover, ground conditions in tunnelling are easily subjected to sudden changes, and therefore, the versatility of soil nailing technique, where length, number and pattern of nails can be easily modified during the excavation progress, represents an additional advantage.

4.3.1 Face reinforcement and fibreglass element

The deformation response of ground and rock masses, subject to excavation action, in terms of "analysis and control" should be carefully studied (theoretical forecasting in the design phase and experimental verification during the actual excavation). The deformation response is complex and involves not just the tunnel cavity, but also the volume of ground that lies ahead of the face, virtually cylindrical in shape, with dimensions quite similar to that of the diameter of the tunnel to be excavated. This region, called the "advance core", is affected by a primary component of the deformation response: "extrusion", which manifests on the surface of the face along the longitudinal axis of the tunnel (bellying or rotation of the face), and "pre-convergence" of the cavity, i.e. the convergence of the theoretical profile of the tunnel ahead of the face (Figure 4.24). These primary components depend on the relationship between the strength and deformation properties of the advance core and its original stress state (Lunardi, 2008).

The deformation of the advance core of a tunnel is the true cause of the whole deformation process in all its components (extrusion, pre-convergence and convergence), and then, as a consequence, the rigidity of the core plays a determining role in the stability of a tunnel in both the short and long terms. The rigidity of the advanced core can be regulated by different technologies, and one of those uses fibreglass elements (GFRP) as a reinforcement of the latter.

The use of fibreglass for the reinforcement plays a determining role in the cavity preconfinement. This material combines properties of high strength with great fragility, which makes it easy to break during tunnel advance using the same mechanical tools as those employed to excavate the ground (Cassani, 2013). The parameters characterizing this intervention are the length, frequency, overlap, cross section and geometrical distribution of the reinforcement. The geometry is defined at the design stage (see example in Figure 4.25) and must be scrupulously complied with at the construction stage in order to ensure maximum effectiveness. Some corrections may be made during construction to adapt the treatment to fit local structural features of the rock mass not detected in the survey phase. This technology can be used in cohesive and semi-cohesive soils and, with a few measures taken to ensure the integrity of the drill holes, even in soils with very poor cohesion. Very good results are proven as well in soft rocks, such as flysch, marls, shales, sandstone and siltstone. If it is well designed and performed, the GFRP reinforcement produces an appreciable improvement in the

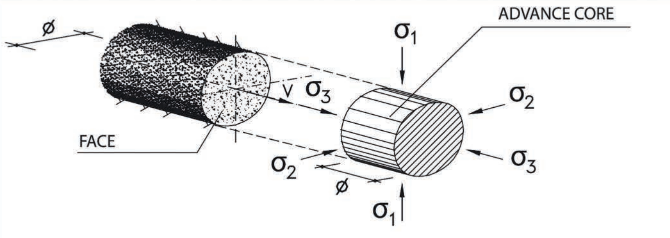
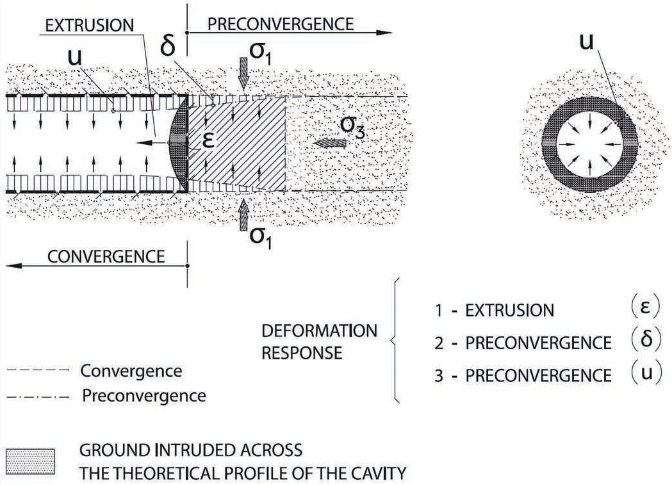
Advance core	
Extrusion and preconvergence (in addition to the already well-known convergence)	

Figure 4.24 The influence of advance core (Lunardi, 2008).

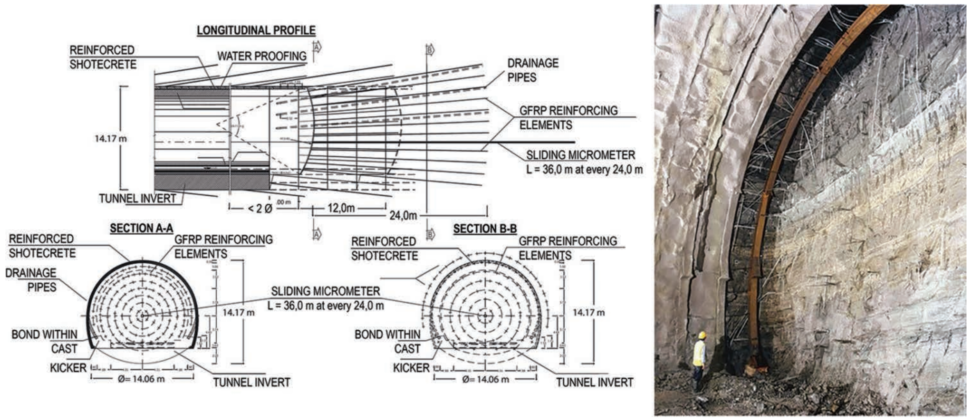


Figure 4.25 Section types and cavity preconfinement after reinforcement of the core-face with fibreglass reinforcement: detail of the reinforced face (TGV Méditerranée, Lyon-Marseilles rail-way line, Tartaiguille tunnel, 1997, ground: clay, max. over-burden: ~ 110 m) (Lunardi et al., 2016).

stress-strain characteristics of the core-face, which makes it a structural element with a predictable and controllable stress-strain response, capable of developing extremely effective cavity preconfinement action.

If needed, it can be combined to advantage with other advance reinforcement techniques such as sub-horizontal jet-grouting, mechanical precutting and pretunnelling.

The GFRPs used for soil nailing at the tunnel face are obtained through an industrial manufacturing process called "pultrusion = pull + extrusion" (Goldsworthy, 1971), which ensures to the final product a uniform distribution of perfectly aligned fibres. In general, the pultrusion of reinforced polymers is similar to the extrusion process used for metals, with the addition of tensile forces applied to the glass fibres to guarantee their alignment before the polymerization of the matrix.

The fibre content, usually expressed as percentage by weight or volume of reinforcing fibres with respect to the total weight or volume, influences the tensile strength of the pipe. It is therefore considered as a reference parameter for the prediction of the pipe mechanical properties. In particular, a higher tensile strength is associated with a higher fibre content, according to a relation that can be assumed as roughly linear. The pultrusion process enables the production of pipes having 60%–65% range in fibre content, which is almost twice the content reached by other manufacturing techniques.

In addition to the tensile strength, the resistance to pull out of the GRFP pipe is another parameter used to identify the soil nail performance. The reinforcing elements are driven or grouted inside drilled boreholes and remain unstressed until the soil movements (for instance, those induced by the tunnel face extrusion) mobilize tensile forces, which are transferred to the ground through friction along the interface. For a better performance, the lateral surface of the pipe needs to be treated so as to improve the adherence with the surrounding mass.

The GFRP reinforcing elements typically used for face reinforcement are the following:

- Structural elements assembled on-site with a diameter between 60 and 100 mm, consisting of three plates (40 mm × 7 mm or 40 mm × 5 mm) fitted on special plastic spacers, or bars of varying length, which can be joined when necessary if care is taken (Figure 4.26).
- Tubular reinforcement of ∅ 60/40 mm and thickness 10 mm (Figure 4.27).

4.3.2 Work phases

The face reinforcement is performed mainly in sections of tunnel with unstable faces or stable faces at short term, and it is absolutely essential for the face to be given a concave shape and lined with shotcrete in order to encourage the channelling of stresses around the tunnel and to guarantee the necessary safety for site personnel.

Work to reinforce the core-face using fibreglass reinforcing elements starts with the drilling of the holes to insert the reinforcement. It is a very delicate phase, and in many cases, it is actually decisive for the effectiveness of the technique. The diameter of the hole must be as small as possible and at the same time allow the subsequent

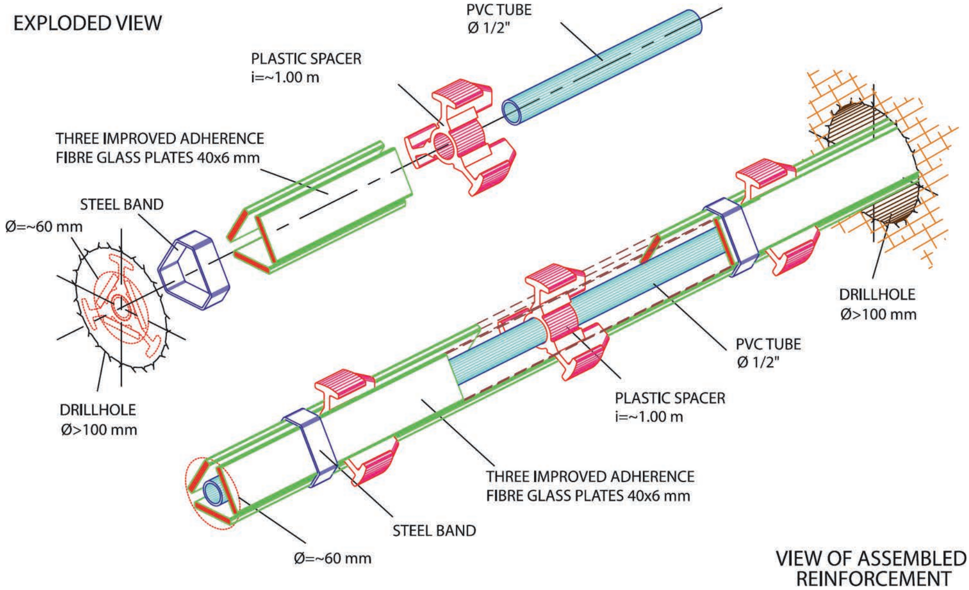


Figure 4.26 Fibreglass structural element for reinforcement of the core-face (Lunardi, 2008).

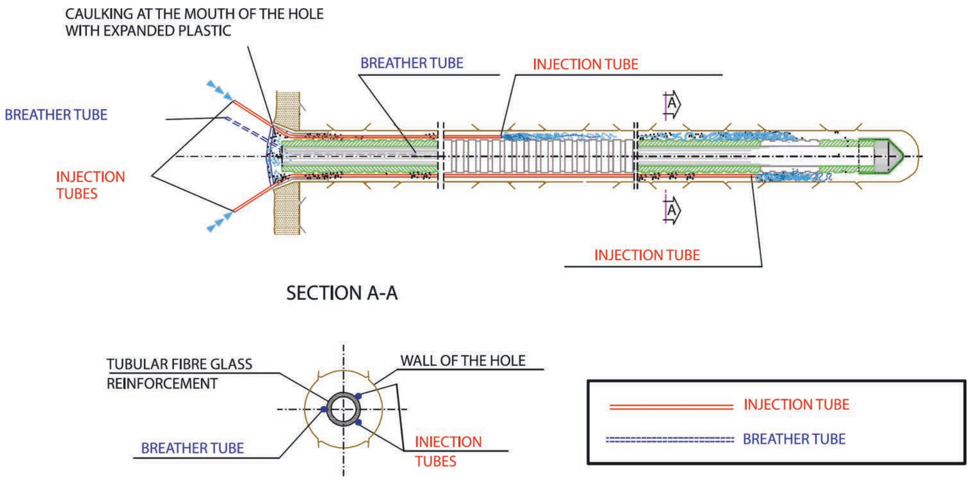


Figure 4.27 Tubular fibreglass reinforcement (Lunardi, 2008).

reinforcement and injection operations to be easily and properly performed. Whichever drilling system is adopted, it must be performed entirely dry, preferably with helical equipment, because water-based drilling fluids would damage the ground around the borehole irreparably (Figure 4.28).

The reinforcement must be inserted immediately after the holes are drilled. In no case whatsoever may more than four or five holes be drilled before inserting the reinforcement (Figure 4.29), because the core-face would be irreversibly weakened.

The reinforcement must be injected with mortar immediately after it is inserted into the boreholes in order to make it active at once (Figure 4.30). The cementation stage also requires particular care, especially when the reinforcement is inclined in an upwards direction. When simple cementation of a structural element in a hole is

Figure 4.28 Holes drilling (Lunardi, 2008).

Figure 4.29 Fibreglass element installation (Lunardi, 2008).

Figure 4.30 Fibreglass element grouting.

required, the mixtures most commonly adopted are made of cement containing additives to accelerate setting times and to prevent shrinkage (e.g. flow cable).

The choice of cement is fundamental for fast setting times, and special test beds should be prepared: the minimum strength after 48 hours should be >5 MPa. Pressure injections are performed using stable cement mixtures with fine or super-aerated cements. The stability of the water/cement mixture is obtained by preparing modest quantities of bentonite to be mixed before the pressure injection.

The fibreglass elements must be cemented using special injection and breather tubes to prevent air bubbles from forming and badly cemented sections as a consequence. It is performed by pumping the cement mix into the hole via a delivery tube until it flows back through the breather tube, which is kept open. At this point, this latter is closed (by simply bending it) and the pressure is raised to up to approximately five bar, which is maintained for a few seconds before pumping is stopped.

Injection must be performed:

- From the end of the borehole towards the wall of the face when the holes are inclined along a downward direction.
- Vice versa, when they are inclined upwards.

In order to prevent injected mortar from leaking from boreholes, the heads of the fibreglass reinforcement must be sealed, with care always being taken to ensure that the breather tube is always positioned above the injection rod. Sealing (which is also referred to as calking) can be performed in three different ways:

- By using rapid setting cement (Figure 4.31).
- By using polyurethane foams.
- By inserting a truncated cone plug of polystyrene in the borehole.

Figure 4.31 Detail of fibreglass reinforcement grouted with cement mixture.

No plates or other devices are needed for sealing. Simple controls must be performed systematically (for each set of reinforcing elements) during construction to check that the treatment has worked properly. The most important are the following:

- the quality of the injection mixture is tested to ensure that it complies with design specifications, with regard above all to how rapidly it sets;
- checking the quantity of mixture injected for each element inserted;
- pull-out tests to check element-mortar-ground adherence and the relative anchoring capacity (cf. Section 4.3.3).

4.3.3 Quality control

The mechanical properties of the reinforcing element and the effectiveness of the reinforcing system (reinforcing element + grouted mixture + surrounding ground mass) must be assessed, respectively, by laboratory test and job site test.

In the following, we will focus the attention on tubular section testing, which represents the most critical procedure. The GFRP profiles have high resistance to tensile force, but low resistance to compression, and this makes difficult performing tests on GFRP hollow sections. The face reinforcement is performed mainly in sections of tunnel with unstable faces or stable faces at short term.

4.3.3.1 Laboratory test

The laboratory characterization of GFRP reinforcing elements is necessary, first, to investigate the properties of the nails and their interaction with the injection mixture, and to highlight the possible better performance of one nail type with respect to others, under prescribed testing conditions, which can be suitably devised to investigate the influence of various factors on the nail performance (Cassani & Zenti, 2016).

Second, the laboratory testing under ideal, controlled and repeatable conditions allows for an accurate and reliable identification of the mechanical parameters of interest that will be taken into account at the tunnel design stage, representing the nails (tensile strength) and the interface layer (bond shear strength).

4.3.3.1.1 FIBRE CONTENT AND PIPE GEOMETRY

As previously pointed out (cf. Section 4.3.1), the fibre content affects the tensile resistance of the profile. The GFRP pipe with improved adherence is traditionally obtained by etching a spiral groove along the external surface (Figure 4.32). The depth of the etching could be equal to 2 mm in a 60/40 pipe, having 60 and 40 mm, respectively, as external and internal diameters. The unavoidable consequence is the cutting of external glass fibres, and the content of continuous fibres actually bearing tensile actions decreases. Therefore, also the pipe tensile strength is reduced with respect to the case of pipe with smooth surface.

Alternatively, the corrugated GFRP pipe is obtained introducing in the production process, before the polymerization of the resin, a phase of preforming. This phase creates a continuously threaded profile, thus reaching the target of improving the bonding adherence while maintaining at the same time the longitudinal continuity of the glass fibres. The depth of the transversal shrinkage is equal to 1.7 mm in the 60/40 pipe.

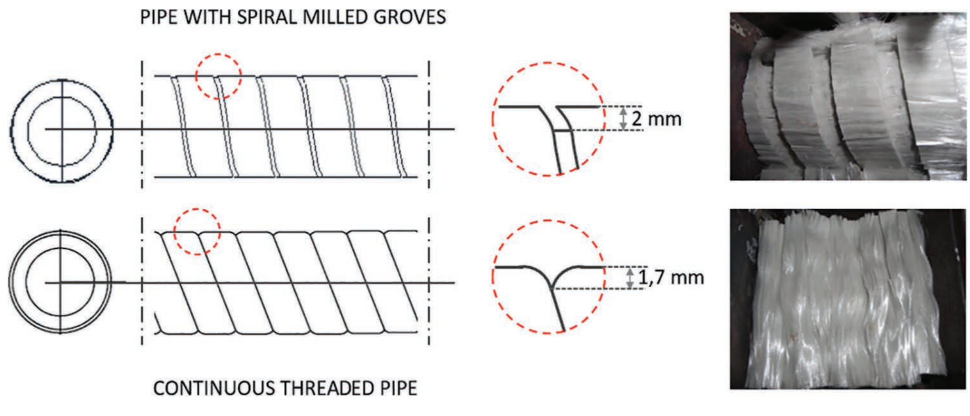


Figure 4.32 GFRP pipe sketch and fibres after heating: pipe with spiral milled groves (top side of the figure); continuously threaded pipe (bottom side of the figure).

The fibre content by weight can be measured after heating treatment, at 800°C for 8 hours. In the case of pipe spiral groove, it is also possible to distinguish between the total content of glass and the content of glass from longitudinal continuous fibres only. In Figure 4.32, the assembly of fibres after heating treatment on the two different pipe samples is shown: the presence of cut fibres is evident in the pipe with spiral milled groves, while in the continuously threaded pipe, the fibres are continuous.

4.3.3.1.2 TENSILE TEST

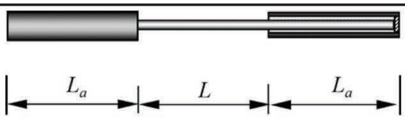
Before the execution of the tensile test, the determination of the equivalent cross-sectional area of GFRP pipe by using the same testing procedure prescribed for the fibre-reinforced polymer (FRP) matrix composite bars (ASTM, 2016) is necessary. The tests consist of immersing the specimen bars (~200 mm long) in a graduated cylinder filled with water and then, once the bars are fully immersed, measuring the volume increase of the liquid. The cylinder must be high enough to prevent overflow once the specimen is immersed. In order to determine the equivalent cross-sectional area of the tested specimen, A_p, its average length, l_p, shall be determined:

$$A_p = \frac{V_l - V_0}{l_p} \tag{4.1}$$

where V_0 and V_1 are the volumes of water in the cylinder before and after immersing the pipe, respectively (Figure 4.33). Once all the equivalent cross-sectional areas of the specimens have been determined, the average value of these quantities, which characterize the geometry of the pipe, may be evaluated.

The tensile test method in the ASTM D7205 Standard (ASTM, 2016) is usually applied to determine the quasi-static longitudinal tensile strength and the elongation properties of FRP matrix composite bars, commonly used as tensile elements in reinforced, prestressed or post-tensioned concrete, but it is easily adapted also to the pipes case. The method requires to equip the specimen by an anchoring system constituted

Figure 4.33 Equivalent cross-sectional areas determination.



FRP bar type	Diameter of the FRP bar, d	Outside diameter of the steel tube	Minimal length of the steel tube, L_a
GFRP	6.4 mm [0.25 in.]	35 mm [1.38 in.]	300 mm [12 in.]
GFRP	9.5 mm [0.38 in.]	35 mm [1.38 in.]	300 mm [12 in.]
GFRP	13 mm [0.50 in.]	42 mm [1.63 in.]	380 mm [15 in.]
GFRP	16 mm [0.63 in.]	42 mm [1.63 in.]	380 mm [15 in.]
GFRP	19 mm [0.75 in.]	48 mm [1.88 in.]	460 mm [18 in.]
GFRP	22 mm [0.88 in.]	48 mm [1.88 in.]	460 mm [18 in.]
GFRP	25 mm [1.00 in.]	48 mm [1.88 in.]	460 mm [18 in.]
GFRP	29 mm [1.13 in.]	48 mm [1.88 in.]	460 mm [18 in.]
GFRP	32 mm [1.25 in.]	75 mm [2.95 in.]	800 mm [32 in.]
CFRP	9.5 mm [0.38 in.]	35 mm [1.38 in.]	460 mm [18 in.]

Figure 4.34 Recommended dimensions of test specimens and steel tubes (ASTM, 2016).

by a steel pipe (Figure 4.34) characterized by a wall thickness of 4.8 mm or more. The standard recommends also a minimum grout space of 4 mm between the outer surface of the bar and the inner wall of the steel tube. The anchor length l_a is the length required to bond the pipe to the steel tube.

The grip zones of each specimen are equipped with a steel tube of 100 m length. To prevent breakage during the test execution, the hollow section of the pipes grip zone is previously filled with the same resin used for the pipe/steel tube bonding. The pipe free length is 100 m. Hydraulic axial loading equipment, with a maximum load at least of 2,000 kN, must be used under a controlled displacement rate, not overcoming the value of 2 mm/min.

In Figure 4.35, the test execution is shown and in Figure 4.36 the specimen after failure. The tests must be carried out at least on five elements.

Figure 4.35 Tensile test equipment and set-up.

Figure 4.36 Glass fibre-reinforced polymer pipes subjected to tensile tests after failure.

4.3.3.1.3 PULL-OUT TEST

Focusing on the assessment of the interface shear strength, the laboratory investigation is customarily carried out by applying a pull-out load, by way of a hydraulic jack, to a pipe driven into a soil model, reconstituted under prescribed conditions, or grouted in a formwork with prescribed grout mixture. Usually, these tests are carried out with the following objectives:

- to assess the response of the reinforcing elements to a tensile load;
- to verify the achievement of a sliding condition, represented by a cumulative residual displacement at the attainment of the maximum pull-out load;
- to identify the parameters for the calculation of the interface bond properties, according to a reference standard.

Among the standards, testing procedures have been defined for the pull out of generic anchors in masonry and in rock (e.g. ASTM, 2004), while for FRP composite bars, the standards basically refer to masonry and concrete (e.g. ACI, 2012). The simplicity and effectiveness of this method suggested designing a test set-up for the standardization of this type of tests when carried out on GFRP pipes.

The pull-out tests are carried out first by inserting the free head of the GFRP pipe in a protective steel cylinder case for a correct gripping and subsequently by applying tensile load. This is necessary in order to extend the gripping surface, thus reducing the risk to damage the GFRP element. The pull-out phase is performed under displacement control, with an applied displacement rate of 0.02 mm/s, with continuous load and displacement monitoring, till the complete pull out (Figure 4.37). The maximum tangential stress at failure τ_{max}, defined as the bond strength, can be calculated as follows:

$$\tau_{\max} = \frac{F_{\text{pull-out}}}{C_b \ell} \tag{4.2}$$

where $F_{\text{pull-out}}$ is the pull-out load at failure; C_b is π times the pipe diameter; and ℓ is the embedment length of the sample within the cement grout block. Considering the

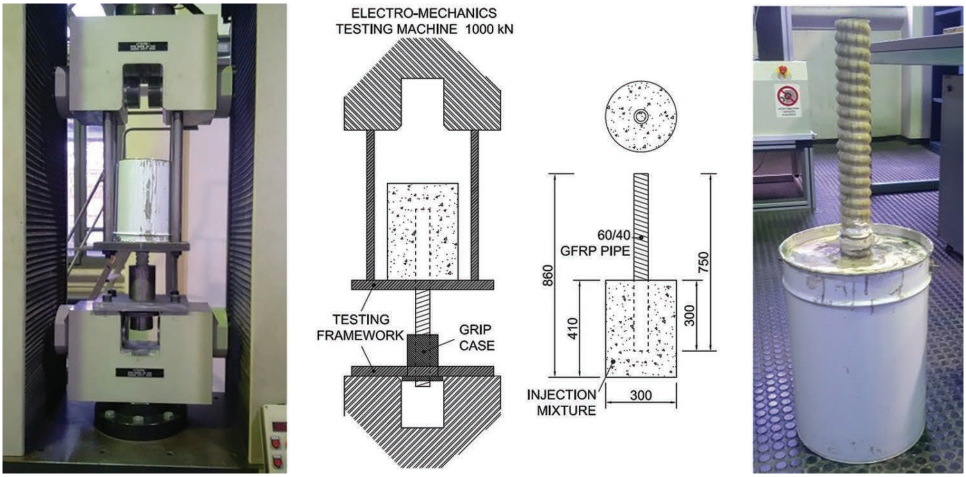


Figure 4.37 Sketch and pictures of the pull-out test equipment and set-up.

diameter of the tested pipes (60 mm) and the embedment length (300 mm), the bond strength is then evaluated assuming the maximum value of load.

4.3.3.2 *Job site pull-out tests*

The effectiveness of the reinforcing system (reinforcing element + grouted mixture + surrounding ground mass) must be assessed by job site test (Figure 4.38). The reinforcing element is inserted and grouted into the borehole so that its head extends from the excavation face. The borehole axis should be driven along the direction normal to the surface, so as to apply a condition of pure traction during the pull out. The grout is a mixture of ordinary cement, water and bentonite, in the ratio 100:60:4 by weight. The head is prepared using a threaded steel pipe, which acts as a protective case and widens the lateral surface of the nail where the load is applied, to reduce the risk of damaging.

The test takes place 24 hours after injection, and it follows a load-controlled procedure, similar to what is prescribed for pull-out tests on rock bolting (ASTM, 2004). Given increments of axial tensile load are applied to the nail by an electrically operated hydraulic jack, and the reaction force is transferred to a stiff steel plate placed against the excavation face (Zenti et al., 2012).

The values of pressure are controlled by an analogic manometer, and the displacement of the head is measured by optic differential levelling. For each load increment, two measurements are made, the first at the application of the load and the second 2 minutes after, with constant applied load. Eventually, the nail is unloaded and, 1 minutes after unloading, the residual displacement is measured, to verify the occurrence of permanent sliding of the pipe along the borehole.

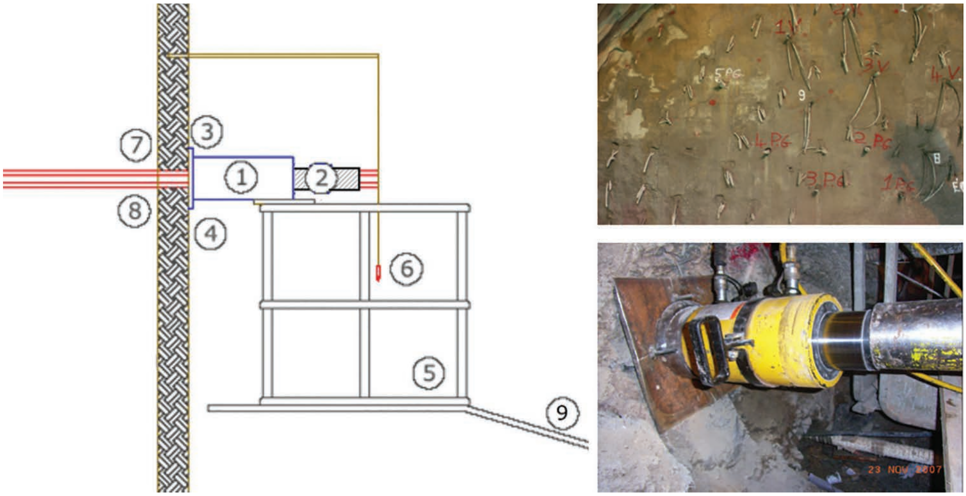


Figure 4.38 In situ pull-out test set-up and devices: (1) hydraulic jack, (2) steel head used as a gripper, (3) stiff steel plate, (4) ball joint, (5) platform for operator, (6) plumb line, (7) reinforcing element, (8) excavation face and (9) mechanical device for platform positioning.

4.3.4 Summary

The use of fibreglass elements is one of the most effective provisions to reduce the tunnel pre-convergence and face extrusion, and its use has become integral part of modern tunnelling. In the recent years, improvements have been introduced in the technological aspects and efforts have been made to better understand the mechanical behaviour of new materials. Tensile strength and pull-out resistance are used to predict the performance of the nail and to take into account the mechanical action of the system at the design stage. They can be assessed by laboratory and field tests.

4.4 GROUTING

4.4.1 Introduction

Grouting is one of the methods for ground improvement used to solve geotechnical problems, such as foundation, cut and cover excavation, tunnelling and seepage control, both above and below the water table. According to Eurocode EN12715 (2000), grouting for geotechnical purposes is a process in which the remote placement of a pumpable material in the ground is indirectly controlled by adjusting its rheological characteristics and by the manipulation of the placement parameters, such as pressure, volume and flow rate. According to FHWA (2001), grouting comprises a variety of techniques employing the injection of a range of materials into soil or rock formations, via boreholes, to alter the physical characteristics of the ground when the materials set. More specifically, grouting can be used to fill fissures and voids in rock, to fill

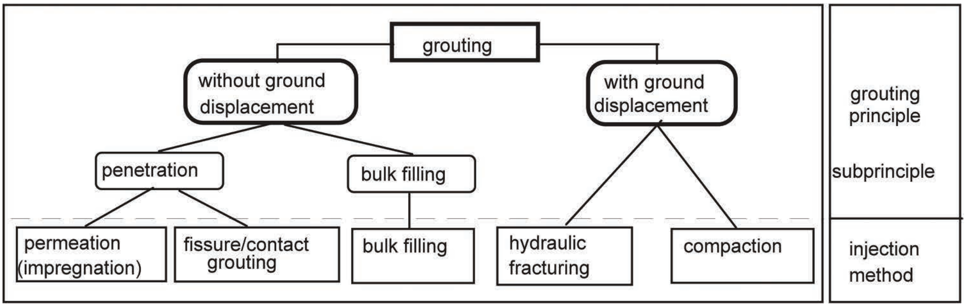


Figure 4.39 Grouting methods (EN12715, 2000).

voids between the ground and man-made structures and to treat fractured rocks and loose formations to enhance strength, impermeability, density and/or homogeneity.

The improvement of the soil characteristics in terms of increasing the mechanical properties and decreasing the permeability may be achieved by different grouting principles, subprinciples and construction methods (Figure 4.39). As far as the grouting principles are concerned, we can consider grouting "without ground displacement" and "with ground displacement".

Grouting without ground displacement entails the substitution of the natural interstitial fluid in the accessible existing voids of the ground by a grout or mortar without any significant displacement of the ground. It includes the subprinciples penetration grouting and bulk filling and the methods permeation (impregnation) grouting, fissure/contact grouting and bulk filling.

Grouting with ground displacement involves the injection of grout into a host medium in such a manner as to deform, compress or displace the ground. It includes two main injection methods: hydraulic fracturing and compaction.

As far as the injection methods are concerned, we can underline that fissure grouting is applicable to rock, permeation, hydro-fracture and compaction grouting to soil (Figure 4.40), and contact grouting and bulk filling to both rock and soil.

4.4.2 Construction methods

Construction methods are usually selected according to the type of soil or rock to be improved, to its amenability to different types of grouts and to the anticipated results as per project's specific requirements. As far as the type of soil and rock is concerned, fundamental is the degree of stability of the grouting boreholes:

- a stable ground (usually rock) may be grouted via uncased boreholes;
- an unstable ground (usually soil or incompetent/weathered rock), on the contrary, requires an injection performed either through special grouting pipes equipped with one-way valves (i.e. tube à manchettes or TAM), or after a pre-grouting to stabilize the boreholes.

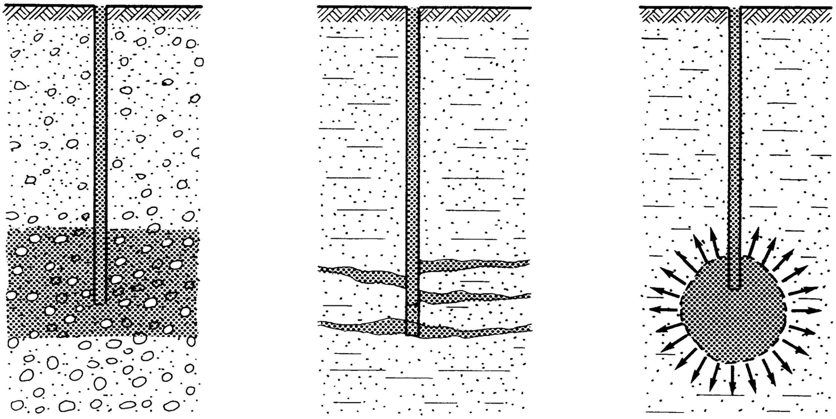


Figure 4.40 Grouting techniques of soils (Manassero, 2002).

4.4.2.1 Penetration grouting

Penetration grouting is a non-displacement method entailing the replacement of the natural interstitial fluid in the accessible existing voids of the ground by a grout or mortar without any significant displacement of the ground. Penetration grouting is therefore the grout injection of joints or fractures in rock, or pore spaces in soils, without producing displacements within the host medium. It includes permeation (impregnation), fissure and contact grouting.

The method entails the following main construction phases:

- drilling boreholes of suitable diameter located according to an appropriate pattern down to the design depth;
- batching and mixing the design grout mixtures;
- injection of the grout mixtures through the soil or rock to be improved, filling pore spaces or fissures.

Penetration grouting is applicable using a wide variety of grout mixtures: pumpable materials (suspensions, solutions, emulsions or mortars), injected into soil or rock, which stiffen and set with time.

Different grouting methods may be adopted for penetration grouting according to the different circumstances encountered (Manassero, 2002).

In rock:

- up-stage method;
- down-stage method;
- Multiple Packer Sleeved Pipe (MPSP) method.

In soil:

- tube à manchettes (TAM) method.

In both rock and soil, grouting can also be performed via the rod string, but it is generally considered only for pre-grouting treatments without particular claims on the anticipated result in terms of improvement.

The up-stage method is applicable to fissured rocks that, however, can assure the stability of the hole for the whole design depth. The hole is drilled down to the required depth in one go. The borehole length is preliminarily divided into stages (usually 2–5 m each) and then grouted, stage by stage, by placing the packer at the top of each stage and grouting all the stages of the hole from the bottom end up (Figure 4.41).

The down-stage method is applicable to incompetent or weathered rocks that cannot assure the stability of the hole for the whole design depth and require stabilization; the presence of incompetent or weathered rock is detected by the hole collapse during drilling or by the improper sealing of the packer at the required depth. The hole length is preliminarily divided into stages (usually 2 to 5 m each). A borehole is drilled down to the bottom level of the first grouting stage and then grouted by a single packer

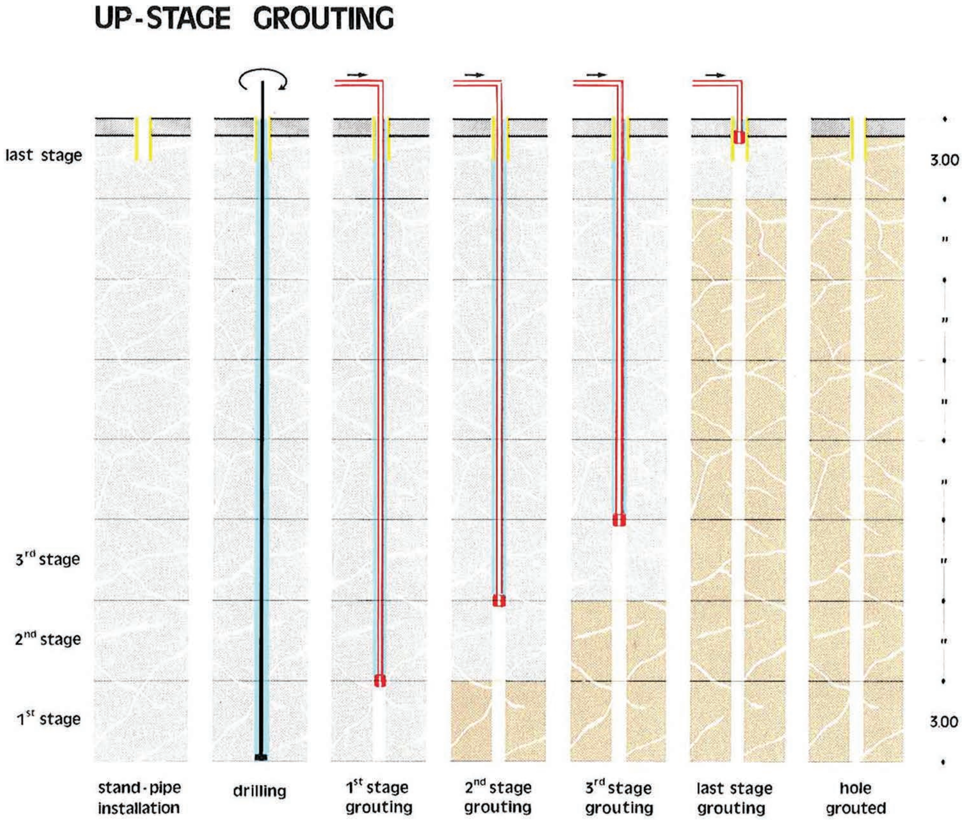


Figure 4.41 Up-stage method, construction sequence (Manassero, 2002).

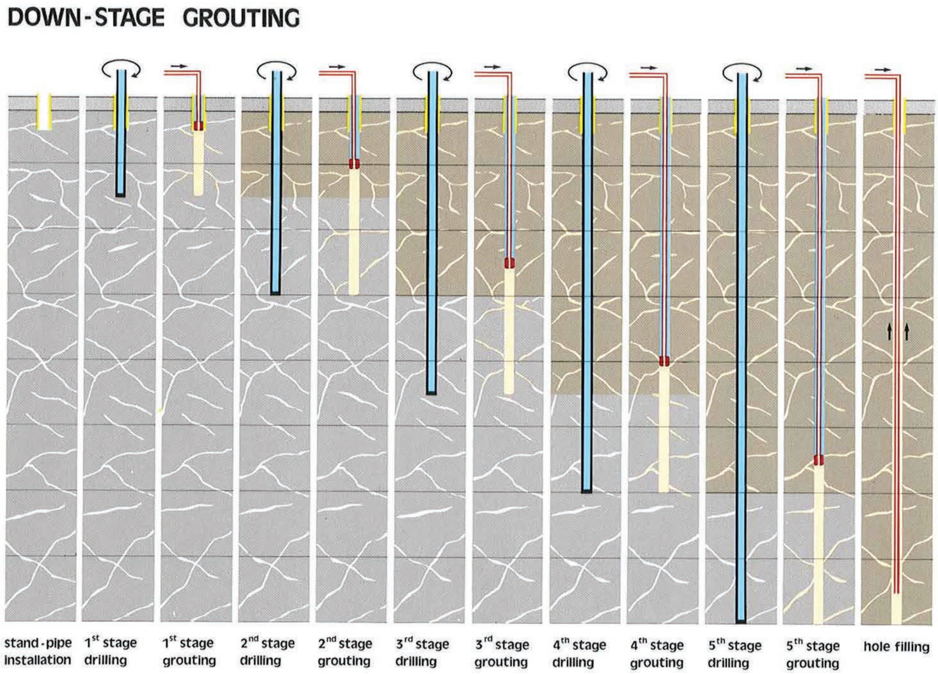


Figure 4.42 Down-stage method, construction sequence (Manassero, 2002).

installed at the top of the stage. At a later time, the hole is extended down to the bottom level of the following stage to perform the next grouting step. This sequence is then repeated until the full depth has been reached (Figure 4.42).

The TAM method is a multiple phase process, which allows several successive injections in the same zone. This involves installing a plastic or steel sleeved pipe (tube à manchettes or TAM) into a grout hole drilled down to the design depth. The pipe is equipped with one-way valves at fixed intervals (0.33 or 0.50 m), each one constituted by small holes, drilled into the pipe to act as outlets for the grout; these holes are tightly covered by rubber sleeves (manchettes), which open only when under pressure and close when pressure ceases. The TAM is permanently sealed in with a sleeve grout composed of a cement-bentonite-water mixture. The sleeve grout seals the borehole between the pipe and the soil to prevent the injection grout from channelling along the borehole. This means that, under pressure, the injection grout breaks through in radial direction and penetrates into the soil. Once the sleeve grout has set, pressure grouting can be performed. In order to inject through a sleeve, a double packer fixed at the end of a smaller-diameter injection pipe is inserted into the sleeved pipe and centred around the sleeve to form a closed chamber with one-way valve outlets (Figure 4.43).

In rock, up-stage and down-stage methods are very efficient, but also very expensive in terms of time and resources. On the other hand, the TAM method normally adopted for grouting loose soils is not applicable to rock because the non-deformability of the material makes it very difficult, often impossible, to break the cement grouted sheath (necessary for injection through the sleeve). For this reason, the MPSP method was developed (Bruce & Gallavresi, 1988); it is a hybrid system worked out

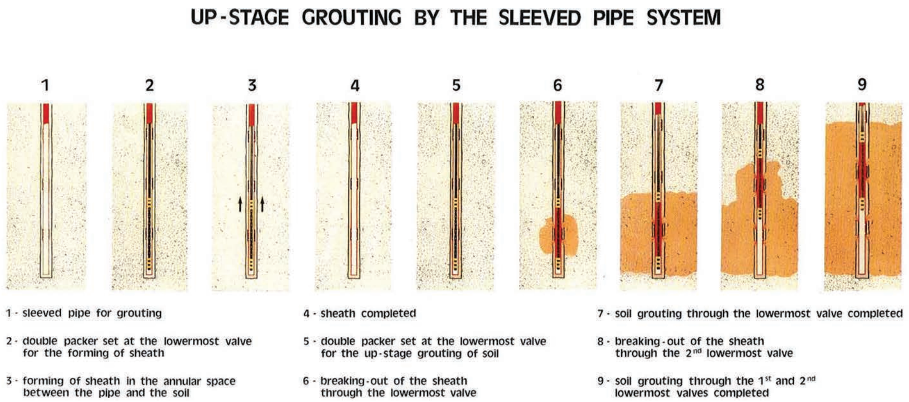


Figure 4.43 TAM method – construction sequence (Manassero, 2002).

to permit the utilization of TAMs (that are very practical and less onerous) also for grouting rock, thus allowing a better industrialization of the grouting process in rock, particularly when the down-stage method is required, which entails many subsequent drilling and re-drilling operations on the same borehole.

The method is particularly suitable to improve grouting operations in those rock formations where:

- caving of the drilled holes and pronounced weathering prevent the sealing off of sections of holes to be grouted at the designed depths;
- the downstage method is not capable of achieving a sufficient consolidation and stabilization of the rock.

The system consists essentially in the installation inside a borehole of a plastic or steel TAM fitted with inflatable bag packers, made by fabric or geotextile, fastened to the pipe at regular intervals (usually 2–5 m), to seal off the hole between which the grouting is to be confined. The sealing action of the bag packers is obtained by expanding them against the hole walls through injection of cement grout into the bags. After the packer grout hardening period, a water pressure test (if required) and, subsequently, up-stage grouting by means of double packer are performed (Figure 4.44).

4.4.2.2 Hydro-fracture grouting

Hydro-fracture grouting is a displacement grouting method, which entails the injection of grout under pressure with the deliberate intent of spatially displacing the host medium. The method is used to increase the density of a plastically deformable material, and the volume of the treated mass where the plastic deformation limit is reached. The ground is fragmented by deliberate hydro-fracturing under a pressure in excess of the local tensile strength and confining pressure, in order to increase total stresses by the wedging action of successive thin grout lenses, to fill unconnected voids and possibly to consolidate the soil under injection pressure. Being difficult to control the propagation of a hydraulic fracture plane, the grouting objective should usually be achieved by an incremental series of injections (repeated and selective), spread over a period of time.

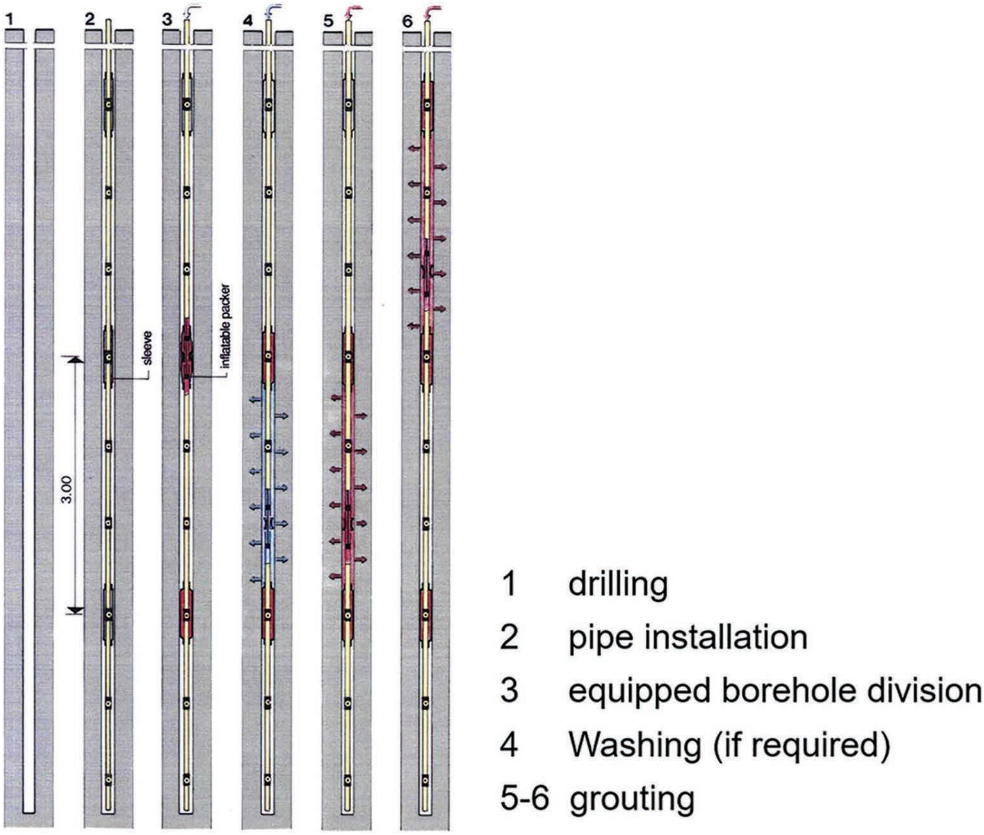


Figure 4.44 Multiple packer Sleeved Pipe method, construction sequence (Manassero, 2002).

The construction method to be adopted to achieve useful hydro-fracturing is the TAM method, the only one suitable to perform an incremental series of injections without re-drilling the grouting borehole.

Grouting by hydraulic fracturing may be used to:

- reinforce or stabilize the ground (soil or rock);
- produce controlled uplift of structures;
- decrease permeability by creating compartments.

Compensation and uplift grouting are particular applications of hydro-fracture grouting that, in combination with an accurate monitoring on displacements (mainly vertical), allows the compensation of possible settlements of ground, buildings and utilities when occurring during tunnelling or digging in general, or their controlled uplift, when settlements occurred in the past.

Both pressure grouting and accurate monitoring on displacements play a fundamental role and are essential to implement this particular technique.

The accurate monitoring on displacements is the "tool" to drive pressure grouting operations, in order to get the goal of a uniform compensation of settlements or the controlled uplift of ground/structures. The monitoring should be carried out by an automatic system, able to measure and record level data continuously and to share them almost "in real time" at least with grouting engineers, who can drive and fine-tune grouting operations accordingly.

Pressure grouting is the "tool" to get compensation of settlements or controlled uplift. Suitable TAM are installed into the boreholes on purpose and sealed into the ground by a sleeve grout. TAMs are arranged in an appropriate pattern, under the foundations of the structure to be protected from excessive settlements or to be uplifted. Through the TAMs, repeated and selective pressure grouting is then performed when settlements occur. The grouting process utilized shall allow numerous repeated injections of grout during compensation and controlled uplift grouting, without creating a build-up of binder around the injection pipes that could hinder breakout pressures; in other words, it could carry out hydro-fracture grouting rather than permeation grouting.

4.4.2.3 Compaction grouting

Compaction grouting is a displacement grouting method to improve the strength and/or stiffness of the soil by slow and controlled injection of a low mobility grout. The soil is displaced and compacted by the grout forced into the soil, as the grout mass grows. Provided that the injection process progresses in a controlled fashion, the grout material remains as a growing bulb within the soil and does not permeate or fracture it. This behaviour enables consistent densification around the expanding grout bulb, resulting in stiff inclusions of grout surrounded by soil of increased density and better mechanical characteristics. The process can be applied equally well above or below the water table. It is usually applied to loose fills and loose native soils that have sufficient drainage to prevent build-up of excess pore pressures (ASCE, 2010).

Compaction grouting, further to compact and densify loose soils, may also be applied to raise and support structures that have suffered settlements. The grout is extruded from an open-ended injection pipe, which is the temporary casing of the drilled borehole. Compaction grouting is usually accomplished up-stage. For each point of treatment, the hole length is preliminarily divided into stages (0.30–1.00 m). A casing is installed tightly in the drilled borehole, with little or no annulus, so that grout is forced to expand radially into the soil and is restrained from travelling upward along the pipe. The open-ended casing is withdrawn up to the top of the deepest stage, and then grouting of the first stage is performed. The casing is then withdrawn up to the top of the next stage, and again grouting is performed, and so on up to the top of the designed treatment (Figure 4.45).

When compaction grouting is used as a means of lifting foundation elements from relatively shallow depths, the up-stage process can be difficult to control, especially when near-surface soils are particularly weak; as an alternative, in these cases, the down-stage method can be adopted. A key factor of the compaction grouting method

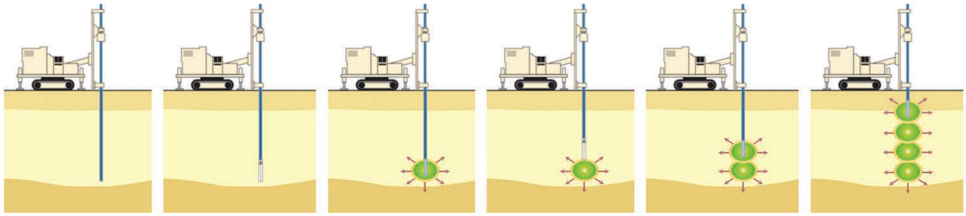


Figure 4.45 Compaction grouting, construction sequence.

are the characteristics of the grout, in order to fulfil three main objectives: (a) pumpability sufficient to enable grout to be injected; (b) behaviour as growing bulb in the ground; and (c) dissipation of any bleed water into the ground. The goal can be achieved by adding in the grout mix the right amount of fine particles and water and the appropriate aggregate gradation.

4.4.2.4 Bulk filling

Bulk filling is the placement of a grout to fill subterranean cavities, both in rock and soil, usually via boreholes on purpose. Void filling usually encompasses one or more of the other grouting methods, and so the grout materials are utilized depending on the purpose and intent of the project. Similar to other forms of rock grouting, the drilling and grouting can be considered an extension of the exploration programme while also remediating the problem (FHWA, 2001). Two main issues have to be addressed while performing bulk filling: (a) the difficulty of completely filling the void and (b) containing the grout within the zone to be stabilized, although the use of low slump grout "barriers", accelerated grouts and grout-filled fabric form can be used to minimize this problem.

4.4.3 Grout mixtures

Grout mixtures may be classified as:

- suspensions (either particulate or colloidal);
- solutions (either true or colloidal);
- mortars.

4.4.3.1 Suspensions

Suspensions are mixtures of liquid and solid materials. They behave as a Bingham fluid during flow, possessing both viscosity and cohesion (yield strength). Particulate suspensions contain particles larger than clay size, while colloidal suspensions contain particles of clay size. Owing to their basic characteristics and relative economy, the suspensions are the most frequently used grouts, both for ground strengthening and for permeability reduction.

Suspensions may be composed of a combination of several raw products:

- hydraulic binders such as cement or microfine cement;
- colloidal stabilizing agents such as bentonite or clay;
- fillers such as fly ash or silica fume or limestone powder;
- admixtures for various purposes, such as plasticizer, dispersant, stabilizer, retarder, anti-washer and sweller.

They are characterized by:

- grain size distribution of the solid particles;
- water/solid and/or water/binder ratio;
- rate of segregation and bleeding;
- water retention capacity/stability under pressure;
- rheological properties and their evolution with time.

Suspensions may be stable or unstable. Stability is generally achieved by means of an increase in the solid content and/or the incorporation of a mineral or colloidal component, often from the bentonite families.

According to the increasing penetrability, suspensions can vary from pure cements (unstable grouts) to cement-bentonite and to cement-bentonite-admixtures suspensions. Penetrability may also be improved reducing the grain size of the solid particles, e.g. replacing ordinary cements with microfine cements. Conversely, to reduce the mobility fillers may be adopted.

4.4.3.2 Solutions

Solutions are liquids formed by completely dissolving a chemical in water to give a uniform fluid without solid particles. They are Newtonian fluids with neither rigidity nor particles and harden in a predetermined period of time, called the “setting time”. They can be true or colloidal solutions. In the case of colloidal solutions, large molecules are contained in the liquid.

Part of the colloidal solutions are the silicate-based grouts, mixtures of sodium silicate and reagent, which change in viscosity over time to produce a gel (gelling process). Sodium silicate is an alkaline colloidal aqueous solution. Reagents may be inorganic (mineral) or organic; the latter ones were often used in the past, but nowadays, they are avoided for environmental reasons.

Inorganic reagents can produce only soft gels, thus being generally used only for permeability reduction; this is because, in order to achieve a satisfactory setting time, the silicate must be strongly diluted. Inorganic reagents contain cations capable of neutralizing silicate alkalinity. Typical inorganic reagents are sodium aluminate and sodium bicarbonate.

Organic reagents would produce either soft or hard gels depending on the silicate and reagent concentration. They cause a saponification reaction that frees acids. Common types include monoesters, diesters, triesters and aldehydes, while organic acids and esters are much less common. A particular kind of silicate-based grout is composed of an activated silica liquor and a calcium-based inorganic reagent, conceived in the 1980s and produced under the name Silacsol-S (Tornaghi et al., 1988) and now available on the market with different denominations according to the different manufacturers. In contrast to commercial alkaline sodium silicates, the liquor is a true silicate solution. The activated dissolved silicate, associated with the mineral reagent, produces calcium hydro-silicates with a crystalline structure quite similar to that obtained by the hydration and setting of cement. The resulting product is a complex of permanently stable crystals. Hence, the reaction is no longer an evolutive gelling involving the formation of macromolecular aggregates and possible loss of silicized water (syneresis). On the contrary, it is a direct reaction on molecular scale. This type of mix has the same groutability range as common silica gels, allowing uniform treatment of medium to fine sands. It has a Newtonian behaviour at fairly low viscosity, up to an effective open time chosen case by case. Afterwards, viscosity increases with time up to the final setting. The activated silicate mix has the stability of cement grouts, thus preventing ground water pollution. A further colloidal solution available is the "nano-silica" grout. It is a pure mineral grout with an extremely high penetration capacity into fine sands, fine cracks and micro-fissures. This capacity is mainly due to a particle size of 0.015 µm (i.e. 15 nm), a viscosity of 5 mPa s and no cohesion. The nano-silica grout is a two-component product: a suspension of colloidal silica (SiO_2) in water and an inorganic reagent. The gelling process of the sol particles takes place by a physical reaction between particles of SiO_2; the reagent acts as a catalyst, and by varying its amount, the open time can be adjusted. The base of the colloidal silica gel is SiO_2 and water, and it is completely safe for the environment.

Pure solutions include various types of resins, such as acrylic, phenol, polyurethane and epoxy. Resins are solutions of organic products in water, or a non-aqueous solvent, capable of causing the formation of a gel with specific mechanical properties under normal temperature conditions and in a closed environment. Resins are used for both strengthening and waterproofing where durability is essential and specific characteristics have to be provided. Their typical field of applicability is shown in Table 4.1.

Table 4.1 Typical fields of applicability of resin grouts

Resin type	*Ground type*	*Use/application*
Acrylic	Granular soil Finely fissured rock	Reduction in permeability Improvement of strength
Phenolic	Fine sand and sandy gravel	Tightening and consolidation
Polyurethane	Large voids	Foaming to block water inflow (aqua-reactive resins) Stabilization or local void filling (two-component resins)
Epoxy	Fissured rock	Improvement of strength Reduction in permeability

Source: EN12715 (2000).

4.4.3.3 Mortars

Mortars are as a minimum made of cement, water and sand. Other products may be included within mortars, such as bentonite, fillers and admixtures. Mortars flowing under their own weight are generally used for filling cavities, large cracks, open fissures and voids in granular soils. They shall be stable, and their rheological behaviour shall be similar to suspensions. Mortars showing high internal friction are used for compaction grouting or for the filling of voids.

4.5 JET-GROUTING

4.5.1 Introduction

"Jet-grouting" is a jet injection technology that has widely been used in geotechnical works since the 1970s. Its first application, in civil construction, was in 1950 by the Cementation Company in Pakistan, and later, it was developed in Japan to refine equipment and methods (Yahiro & Yoshida, 1973). In Italy, jet-grouting was used since 1990s in foundations and underground works (Lunardi, 1992, 1997; Sanella, 1991). Nowadays, it is one of the most popular ground improvement techniques due to its applicability in almost all soil types. Its peculiarity is to change drastically ground characteristics, thanks to the effect of very high-speed jets, able to break up the soil and to mix it with cementitious slurry, improving soil's mechanical parameters and reducing its permeability. Several techniques and construction processes can be used for jet-grouting execution, depending on the types of fluid injections and on working parameters; in the following, a general overview of the technology will be given, together with the design criteria for the more frequent geotechnical applications. A code reference is in Eurocode EN12716 (2000).

4.5.2 Jet-grouting technology

The basic concept of jet-grouting technology is schematically represented in Figure 4.46a: a high-speed fluid (water jet or grout jet) is injected through small-diameter nozzles into the subsoil to erode the surrounding soil, while the nozzles are rotated and lifted towards the ground surface at a constant speed. The eroded soil is simultaneously mixed with the injected grout to form the admixture, and a soil-cement column with a quasi-cylindrical shape would be formed after some days of solidification. It is possible to reach columns with diameters ranging between 500 mm and up to 2,500–3,000 mm, owing to the adopted systems, the working parameters and the soil types. The operations to be carried out are performed in two stages (Figure 4.46b): (a) insertion of the drill rods, equipped with the nozzle, down to the design column depth by drilling. To achieve good results, this stage requires skill, as the treatment could be compromised by unexpected rod deviations. (b) Return or extraction stage: the drill rod is extracted with rate of ascent and angular speed, carefully controlled while injecting fluid (grout, water, air, etc.) through the nozzles. Grout pressure, rate of ascent, angular speed, and number and diameters of nozzles are the main parameters to be

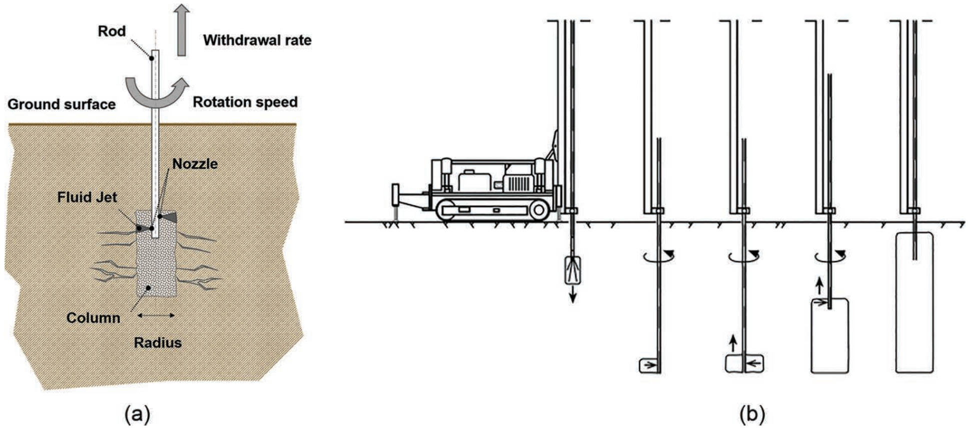


Figure 4.46 Jet-grouting (a) Schematic view of the technology; (b) operation stages.

controlled to obtain the improved soil of the desired shape and size. Soil investigation and field tests must be carefully planned for each application.

4.5.2.1 Executive systems

Based on the different methods of fluid injection, jet-grouting technology can be conventionally classified into three basic types of systems: (a) single-fluid system (only grout), (b) double-fluid system (grout and air) and (c) triple-fluid system (water, grout and air). The "single-fluid" system utilizes grout as the cutting jet to achieve cementation of the eroded soil. In the most common case of drilling without casing, a self-drilling monitor will be used, by rotation or roto-percussion system, with the drilling rod equipped both with injection nozzle and with cutting tool. In the "double-fluid" system, a compressed air shroud is introduced around the grout jet to enhance the cutting distance of the grout jet; the construction process is the same of the single-fluid system, adding an additional nozzle for the air. In the "triple-fluid" system, water is used for the cutting jet together with a compressed air shroud, and grout is injected separately through a lower nozzle at a much smaller pressure to mix with the eroded soil. The adoption of a lower viscosity fluid such as water (in comparison with that of grout) allows the cutting distance to be further enhanced, especially in cohesive soils. For double- and triple-fluid systems, only the rotation drilling system is suggested to prevent damages of the two- to three-way rods. The three methods are represented in Figure 4.47. An alternative system to the triple-fluid system is represented by the "two-stage" system. Grout injection is preceded by very high pressure water injection (first stage), designed to produce preliminary breakdown of the ground; the following grout injection (second stage) replaces water and the washed-out finer soil fractions.

With respect to the "permeation" grouting technology, the jet-grouting system provides a greater ratio between grouting and soil volume, so that waste materials, with cement grout too, get out from the drilling hole. The control of the quantity of the waste material is very important for the quality and the dimensions of the jet-grouting

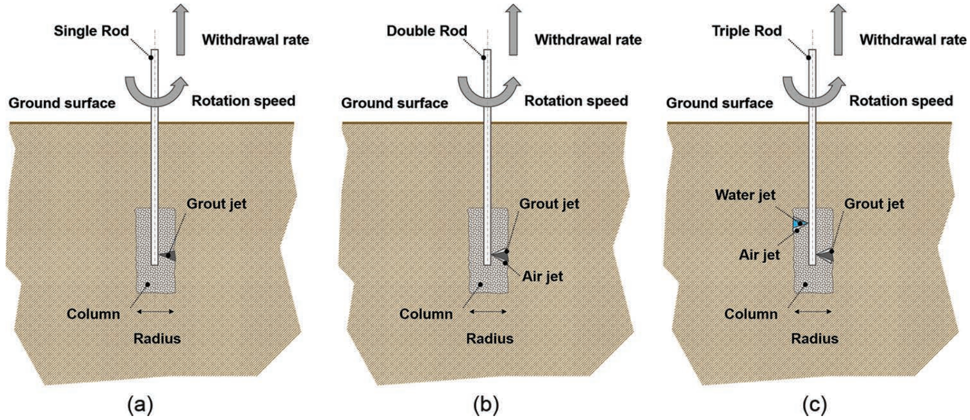


Figure 4.47 Illustration of conventional jet-grouting systems: (a) single fluid; (b) double fluid; (c) triple fluid.

columns and for the impact on the surrounding ground (such as lateral displacement and ground heave), especially in cohesive soils Eurocode EN 12716 (2000).

The described types of system were developed between the 1970s and the 1990s, around the world. In 2000s, in Japan and in the USA, the "Super Jet" technology was introduced to produce larger jet-grouting column (Brill et al., 2003); the system is based on the conventional double-fluid system, using two opposing nozzles to eject high-pressure grout (30 MPa) shrouded by compressed air (0.7–1.05 MPa). Compared to the conventional double-fluid system, the "Super Jet" technology has a higher injection volume of grout flow in construction, and hence, the diameter achieved by the Super Jet technology can reach in some cases about 5 m (Essler & Yoshida, 2004).

Recently, some improvement of the technology has been developed to achieve quick solidification of soft soils by jetting with two types of binders, cement-slurry and "sodium silicate" as an accelerator. This technique is called "Twin-Jet method" (Wang et al., 2013), and it is particularly advantageous in horizontal jet-grouting applications, especially under groundwater level in order to prevent the soil water pressure washing out the cement slurry during injection. To achieve a quick gel of soft ground after jet-grouting, the hardening process of an admixture of grout-soil must be accelerated by adding a binder (silicate/water-glass) into the admixture of grout-soil, gelling in few seconds. The Twin-Jet method is developed by starting from the traditional triple-fluid system (Kim, 2008); the high-pressurized grout shrouded by compressed air is jetted out to erode the soil, and the admixture of grout-soil is formed. Then, the accelerator solution shrouding the high-pressurized grout is jetted into the admixture for a quick gel, and a jet grout column can be formed after hardening. A similar approach is used in the system called "Quick-Set Jet – QSJ System" (Sanella, 2019); the system has successfully been applied in tunnel works for the execution of horizontal jet-grouting columns under water table. The QSJ system is based on the single-fluid system, adding a nozzle for silicate injection together with the nozzle for grout injection. A preventer system is adopted at the drilling start, to prevent any leakage. Finally, jet-grouting technology has also been successfully applied to the waterproofing of rocks where traditional low-pressure injections are inefficient as the injected grout tends to escape through fractures and joints without filling them sufficiently; the working parameters must be carefully calibrated to fit the system with the rock mass fracturing.

4.5.2.2 Equipment

In Figure 4.48, the equipment required to perform jet-grouting treatments is shown: (1) cement bins, (2) batching plant, (3) 500–2,500 hp high-pressure pump, with compressors, (4) panel board and control board and (5) drilling rigs. The string of drill rods on the rig has joints that withstand high pressure; at the bottom end, it is connected to the injection chamber and to the bit, usually a tricone. In the single-fluid system, the upper rod is connected to the high-pressure grout pump by a swivel joint and a flexible pipe (6). Standard rod strings of 60, 76 and 90 mm o.d. are used. Equipment used to perform double-fluid and triple-fluid jet-grouting is provided with a swivel for separate grout-air and grout-air-water supply, respectively, fed by appropriate pumps and compressors. The rods used for double-fluid jet-grouting (76–90 mm o.d.) have two coaxial ducts to allow separate flow of air and grout. The triple-fluid system requires triple-duct rods in order to allow air, water and grout to flow separately. The injection chamber is located just above the bit. For the single-fluid system, it is a hollow steel cylinder, about 400 mm long, usually of the same outer diameter as the drilling rods. The walls have one or more radial holes fitted with nozzles, and there is a central neck at the bottom of the chamber, 20 mm in diameter, to allow the drilling fluid to reach the bit when drilling. This neck can be blocked by dropping a steel ball (22–25 mm in diameter) inside the rods, thus forcing the grout to be injected through the side nozzles. Grout nozzles can be one to four in number, with inner diameters generally ranging from 2.2 to 6.0 mm; they are usually staggered 1–2 cm, from each other.

Drilling operations are usually performed by rotation or rotary-percussive action, with or without the use of drilling fluid. Rotary drilling, requiring light drill rigs, is preferred in medium- to fine-grained soils, while in cohesionless coarse-grained soils

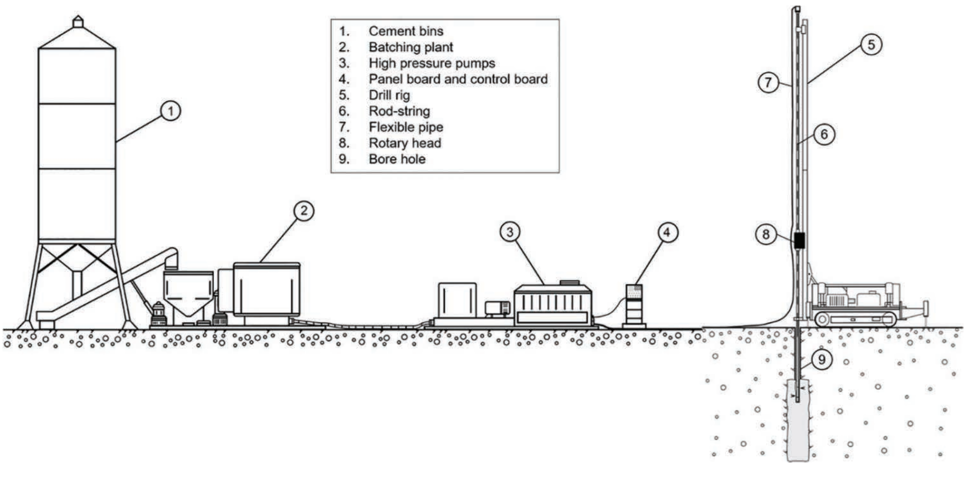


Figure 4.48 Equipment required to perform jet-grouting treatments (Lunardi, 1997).

or when large boulders are found, rotary-percussive drilling can be better in terms of performance, although heavier equipment is required (not suggested for double- and triple-fluid systems). The same rig is generally used for drilling and high-pressure grouting. Once drilling has finished and the required depth is reached, the grouting stage can start. The pump works at a very high pressure (40–70 MPa) pushing the grout through the flexible pipe into the rods, down to the injection chamber and, finally, through the nozzles. The fluids are pumped into the ground at very high energy levels (the jet penetrates the ground at a speed of 800 km/h or more). Special drilling systems have been studied to perform horizontal jet-grouting in underground constructions.

4.5.2.3 Working parameters

The key jet-grouting operational parameters governing the jetting performance are as follows:

- Characteristics of jetting fluid, i.e. water-cement ratio of grout (W/C).
- Pressure (MPa) and flow rate of jetting fluid ($Q = m^3/h$).
- Jetting time, which is a function of the traverse velocity of nozzle, and hence the withdrawal rate (m/min) and rotation speed (rpm).
- Characteristics of nozzle: nozzle diameter (mm), number of nozzles (N) and nozzle shape.

The injection pressure is controlled by pressure gauges; the jet energy and consequently the radius of action mainly depend on pressure. The upper pressure limit is essentially determined by the capacity of the pump used. The number and diameters of nozzles determine the injection capacity: the volume of grout injected into the ground per unit of time and, consequently, the rate of treatment. High flow rates require high-power pumps to maintain high pressure. Larger nozzle diameters make more efficient use of the power employed, while a large number of nozzles, with the same rates, decreases performance, due to a greater loss of head, so it is preferable to limit the number of nozzles. The water-cement ratio of the grout is the most important parameter regarding the mechanical properties of the treated soil and the initial behaviour of the soil-grout mixture; a low water-cement ratio is extremely important where there is groundwater flow, as this could wash away the cement shortly after injection. The injection time depends on the withdrawal rate and the angular velocity of the drill rod; it is controlled by a timer placed on the drill rig (normally, 1 m every 2–6 minutes and 10–20 rpm). Raising is usually performed in 4–5 cm steps, thus allowing the jet to act on the surrounding ground, for set time intervals. The diameter and the mechanical properties of the ground treated, as well as the time required for treatment, are strongly affected by the withdrawal rate.

Table 4.2 shows the range of jet-grouting parameters commonly adopted for the three conventional jet-grouting systems (Lunardi, 1997a). The applied fluid pressure for cutting jet ranges, in general, from 30 to 70 MPa for the single- and double-fluid systems. In the triple-fluid systems, typical injection pressure for the water cutting jet is 30–40 MPa, while the grout is introduced of a much lower pressure of 7–10 MPa and is just used to mix with the soil eroded by the high-pressure water jet. The traverse velocity of the nozzle in the triple-fluid system is smaller than that in the single- and

Table 4.2 Ranges of jet-grouting parameters for conventional jet-grouting systems

Parameter	*Single-fluid system*	*Double-fluid system*	*Triple-fluid system*
Water pressure (MPa)	–	–	30–40
Flow rate of water (L/min)	–	–	80–200
Number of nozzles (n°)	–	–	1.5–3.0
Nozzle diameter (mm)	–	–	1–2
Air pressure (MPa)	–	0.7–1.5	0.7–1.5
Flow rate of air (m^3/min)	–	8–30	4–15
Grout pressure (MPa)	40–70	30–70	7–10
Flow rate of grout (L/min)	100–300	100–600	120–200
Grout density (g/cm^3)	1.25–1.6	1.25–1.8	1.5–2.0
Number of nozzles (n°)	1–6	1–2	1–3
Nozzle diameter (mm)	1.0–4	2–7	5–10
Withdrawal rate (cm/min)	15–100	10–30	6–15
Rotation speed (rpm)	7–20	2–20	7–15
Specific energy (MJ/m)	10–20	40–80	120–200

double-fluid systems. Typical withdrawal rates, expressed in cm/min, and rotation speed, in rpm, are reported in the table. It is possible to link the flow rate of jetting fluid (Q), the volume (V) and the speed of treatment (v), with the formula $V = Q/v$; the flow rate Q depends on jetting area, i.e. the number and diameters of nozzles, and on the jet speed, in turn related to the injection pressure (P). Thus, the knowledge of these working parameters allows to evaluate a global design parameter: the "linear specific energy", by the formula $E = P\ Q/v$, expressed in MJ/m of linear jet-grouting, or the "volume specific energy", by the formula $E = P \cdot Q/(v \cdot V)$, expressed in MJ/$m^3$ (according to Tornaghi, 1993). Using these correlations, it is possible to dispose of a rational approach to compare several combinations of working parameters and find the best technical and economical solutions for each application. The common value of the linear specific energy is in the range 10–20 MJ/m for the single-fluid system (considering column's diameter in the range 600–1,000 mm), in the range 40–80 MJ/m for the double-fluid system (column's diameter in the range 1,400–2,500 mm) and finally up to 120–200 MJ/m for the triple-fluid system. In Tornaghi (1993), a detailed description of the parameters governing the specific energy is reported, together with some predictions about grouted volume and diameter in the most ordinary contexts. By using these bibliography data, the "specific energy" approach can be a useful tool for the design of jet-grouting treatments.

4.5.2.4 Control procedures

During grout injection operations, grout samples will be regularly taken, to control the "grout quality" (composition and grout characteristics). The routine controls include the following:

- Density test
- Marsh viscosity test (recommended about 40–45 seconds)

- Bleeding test (recommended <5% at 2 hours)
- UCS test (28-day strength)

The frequency of the grout tests should be defined in the design stage, recommended at least one in a day (for each column during filed tests execution). The drilling and grouting phase should be automatically monitored by a device on the drilling rig (i.e. Jean-Lutz system or similar). During the grouting phase of each column, the following parameters must be monitored and recorded by the automatic device: grout pressure, grout flow rate, volume injected, speed rotation and withdrawal rate. The data of each treatment will be noted, together with the hole location and reference number, the date and time of grouting operation, the grout mix identification and any other relevant observation (waste materials, injection's stops, etc.), to prepare an "identity card". It's very relevant to note the drilling deviation in depth too, especially for waterproofing treatments at large depths; a specific tool, i.e. "Tigor System", could be fixed at the top of the rig, to record the deviation of the rods during drilling. These data are very useful to point out any holes in the treatment's thickness, so as to plan additional columns. About the planning of drilling and grouting's activities, consider that no drilling activity can be carried out within 2.0–3.0 m around a hole under grouting; in the execution sequence, drillings will be done alternatively, in order to let the executed column hardened.

4.5.3 Application's context and soil's improvement

From the experiences acquired in several geotechnical works, we can state shot jet-grouting can be successfully performed in any type of soil (Figure 4.49a), independently of grain size and permeability, with the exception of very hard cohesive soils whose strength cannot be overcome by the jet (such as stiff silts and clays, with undrained cohesion greater than 0.2–0.5 MPa). This technology has the advantage of being able to treat "stratified soils" too (alternating sands, silts, clays, etc.), providing almost uniform levels of cementation and waterproofing. In fine-grained soils, the outer surface of the columns obtained is normally well defined and fairly regular; in coarse-grained and heterogeneous grounds, this surface is irregular and there is the systematic appearance of "root effect", i.e. grouting through "claquage" outside the radius of action of the jet. The occurrence of still groundwater does not in any way compromise the results of the treatment. When seepage occurs, special precautions, such as the addition of accelerators to the grout, give good results even at the groundwater speed of the order of 0.1 cm/s. The compressive shear strength of jet-grouted soil generally increases from clays to gravels (see Figure 4.49b). Maximum strengths of about 20–30 MPa could be reached in sands and gravels, while in fine-grained peaty soils, it is hard to attain values ten times lower. With single-fluid systems and grout composed of cement and water only (C/A ~0.8–1.2), the long-term strength in sandy and gravelly soils usually ranges between 10 and 20 MPa; in fine-grained soils, the strength values are in the range of 2–6 MPa. The stiffness of grounds treated with different jet-grouting techniques presents a wider variation; in general, the ratio E/R between the secant modulus E and the compressive strength R tends to increase with R, from a minimum of 200–300 in fine soils to about 1,000 in gravels and sands. The strength and stiffness

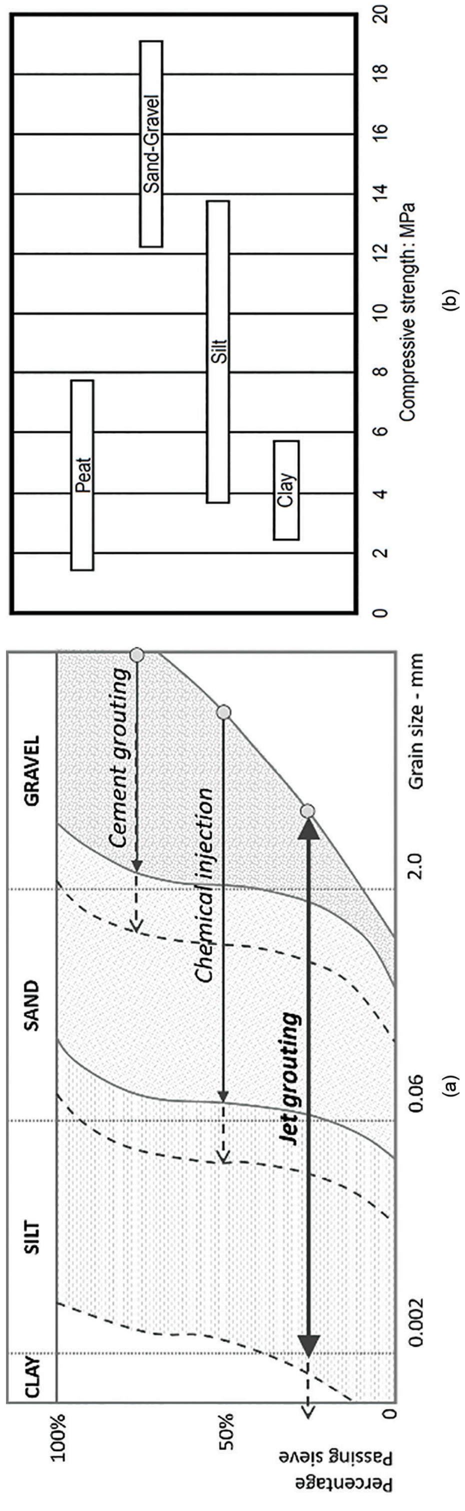


Figure 4.49 (a) Jet-grouting applicability; (b) compressive strength of jet-grouted soils.

of soil treated by double-fluid system are slightly penalized by the possible occurrence of air bubbles.

4.5.4 Design criteria

The design of ground improvement using jet-grouting must involve these main following stages: (a) defining soil investigations and preliminary field tests, (b) choice of grout type and operating parameters, (c) deciding the pattern, shape and size of the grouted volumes, (d) identification of the most appropriate numerical model to study the stress-strain distribution in the soil treated and (e) choice of the monitoring and controlling systems.

4.5.4.1 Soil investigation and field tests

Preliminary investigations are very important to assess both jet-grouting's feasibility and efficiency and to define the main working parameters. Preliminary investigations include: (a) exploratory boreholes to determine ground conditions and stratigraphy; (b) static/dynamic penetration tests to evaluate the mechanical properties and stiffness of the ground; (c) laboratory and in situ permeability tests, particularly for cohesive soils; and (d) field injections tests for deciding on the grout composition. A number of test columns or treated volumes of the same size and pattern as the actual design are drilled and injected on a suitable site, generally adjacent to that of the actual operation, so as to execute a "field test". The working parameters are varied for each test column, or each group of columns, in order to choose the most appropriate combinations based on the test results. Upon completion of the field injection tests, the following tests are performed:

- Destructive tests on cores of treated ground, both vertically and transversely cored, to evaluate the continuity of the treatment (RQD recommended >70%) and to take specimens to evaluate mechanical strength in laboratory tests.
- Sonic tests (downhole and cross-hole). Vertical pipes are inserted into the ground and into jet-grouted volumes at preset distances. In downhole tests, a probe equipped with a sound transmitter and a sound receiver device is lowered into the borehole. The sound waves emitted pass through the treated ground surrounding the hole before reaching the receiver. Amplitude, frequency and shape of the signal are profoundly modified by the characteristics of the material, which also affects the speed of the sound. By measure of the cross-hole technology, the horizontal wave velocities between two points located in adjacent boreholes by inserting a sound transmitter and sound receivers are measured. The recorded data allow us to cross-check the homogeneity of the treatment and the stiffness of the grouted soil.
- Direct site inspection of treated volumes, by excavation, to visually check diameter, structural continuity and possible overlapping (Figure 4.50).

A further method to check the geometry of the test columns, especially at great depth where direct inspection is not possible, is the use of "painted bars", drilled into the

Figure 4.50 (a) Direct site inspection; (b) coring of treated soil; (c) cross-hole tests.

soil at preset distances from the grouting hole; if, during the injection stage, the grout jet reaches the painted bars, removes the colour so that, once completed the injection, pulling out the bars is possible to define the actual treatment's diameter. A similar approach could be used by drilling into the soil two/three PVC pipes equipped with acoustic sensors; if the grout jet reaches the sensors, vibrations are recorded to define the radius of jet penetration (this method has been patented as "ACI Acoustic Column Inspector" by Keller Grundbau Ges.mbH). Finally, thermometric system could be used; the method consists in interpreting the variation over time of the temperature measurements acquired automatically at the centre of the jet-grouting treatment, detected by means of a thermometric column consisting of one temperature sensor every 30 cm depth, installed at the end of the injection phase. The interpretation is based on the thermochemical parameters of the cement, and on the exothermic nature of the hydration process of the mass of soil/cement, allowing to define the radius effectiveness of the column and the quantity of cement.

If the jet-grouting treatment is aimed at guaranteeing the waterproofing of underground works or shafts, such as plugs or retaining structures, it could be very useful to execute a trial mini-shaft, and to perform a pumping test inside (step-drawdown tests) to evaluate the watertightness of the system. Some piezometers are installed inside and outside the mini-shaft, at different distances from the pumping area, to monitor the effects of the pumping well: theoretically, outside the shaft, the groundwater level will be undisturbed, while inside the box, a depression cone will grow initially from the pumping well reaching later the jet-grouting wall. The results will allow to evaluate the effectiveness of the waterproofing treatment by jet-grouting or to point out the presence of leakages. In order to identify the location of the defects, "thermometric" tests should be planned: some vertical boreholes have been placed all around the plug to monitor the variation in the temperature of the ground; some boreholes are equipped by wells. During the curing of the cement, the release of heat from hydration leads to an increase in ground temperatures in the areas surrounding the sealing elements. However, if, during dewatering by the wells, water flows into the excavation, this will change the temperature profile in the ground around the affected area. The ground temperature therefore adjusts to the temperature of the inflowing water; the cooled area extends to the area affected by leakage. Temperature measurements in sealed excavation pits therefore enable the location of leaks in the sealing system. These methods have been used for the extensive jet-grouting treatments of the works reported in Lunardi et al. (2019) (Figure 4.51).

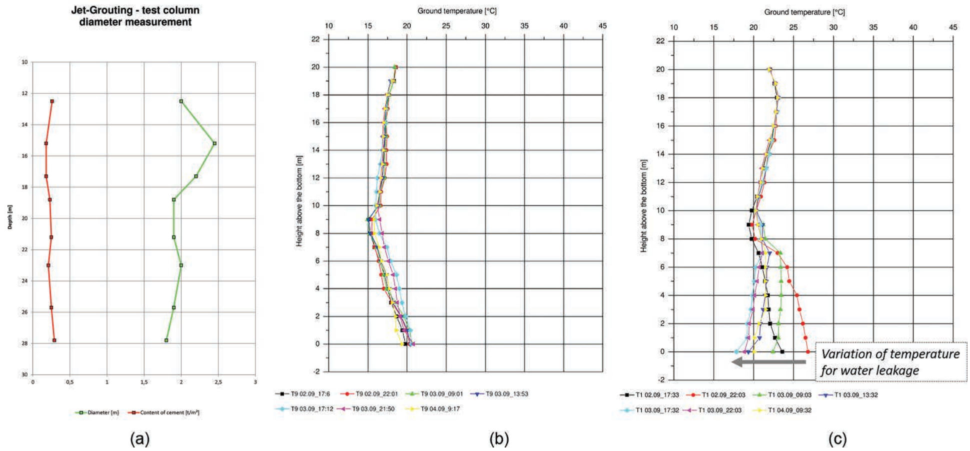


Figure 4.51 Thermometric test for column diameter measurement.

4.5.4.2 Design approach and technical specifications

To define the pattern and the dimensions of jet-grouting treatments, it is essential to understand that ground volumes treated by jet-grouting are cemented ground and not real structures; there phase to define thickness and required strength, it is necessary to predict the stresses acting on treatments. The treated ground can take compression and shear, and consequently, the application of other types of stresses, i.e. tension, should be avoided. In this case, the insertion of bars and steel sections into improved ground can be achieved, especially for retaining structures, usually before setting takes place, by gravity or using high-frequency vibrators or re-drilling. It must be remembered that the existence of more rigid volumes in the ground may possibly be used to produce a system channelling stresses along a desired direction and these stresses should be carefully considered in dimensioning. The analysis of such problems implies generally the use of the finite element method, since the types of problems connected with jet-grouted ground are clearly non-linear. In addition to this, it is often necessary to analyse structures whose shape changes over time due to excavation or construction operations. This requires starting with the analysis of the natural untreated conditions and passing through an appropriate sequence of stages realistically approximating the actual development of the work. Once the geometry of the treatment is defined and the required strength is fixed, it is necessary to study the correct pattern, considering the column diameters easily achieved in the geotechnical context of intervention and an appropriate overlapping to ensure the continuity of treatment. The working parameters will be set, based on database experiences in similar contexts or according to the specific energy approach discussed in Section 4.5.2.3, and they will be confirmed or fine-tuned by a field test according to the specifications described in the previous chapter. Designer must define in detail, in addition to the executive technical specifications, the prescribed requirements in terms of diameter, strength and permeability of the soil treated, to be checked by tests.

4.5.4.3 Monitoring systems

It is important to organize an efficient system for monitoring jet-grouting operations, during treatment and after treatment. In addition to the monitoring of the execution data described in Section 4.5.2.4, it is necessary to detect possible surface lifting and/or settlement of the adjacent ground (especially when the work is carried out near buildings or other structures) and to analyse the effluent material flowing to the surface during injection. After treatment, monitoring includes load tests (in the case of foundation columns), sonic tests to check the improvement of mechanical properties as well as the continuity and possible overlapping of contiguous columns, the taking of samples to test the UCS strength and permeability tests to cross-check the watertightness.

4.5.5 Projects' applications

The jet-grouting ground improvement technique has widely been used in several fields of civil engineering, such as retaining and foundation work, slope stabilization, hydraulic works and excavation of tunnels in loose ground (Lunardi, 1997). Several applications of jet-grouting are referred to retaining structures, open shafts, cut and cover and trench excavations, or execution of plugs for waterproof excavation under water table. These techniques are applied for the construction of tunnel portals in cohesionless or slightly cohesive soils, too. Tunnelling can be performed even in case of a very low overburden, by jet-grouting shell, arch-shaped, thus minimizing the risk of slope instability and providing outstanding results from an environmental and landscape point of view. Design schemes are reported in Figure 4.52, and case histories are in Figure 4.53.

Designs consisted of one/three rows of columns of treated ground spaced at varying distances, located along the edge of the future portal excavation, sometimes with a "strut-and-tie" scheme (Lunardi et al., 2014). When tunnels start in critical geomorphological contexts, jet-grouting treatment, fan-shaped, "buttresses" (jet-grouted columns radially placed, in the plan view, around a circular arc) or large-diameter shafts are often very efficient.

Jet-grouting has also a fundamental role in the progress that has been achieved over the last 30 years in tunnel construction (Lunardi et al., 1986). Jet-grouting treatments can be executed by surface, where space is available and overburden is limited, or directly at the tunnel face (see Figure 4.54). Horizontal jet-grouting made it

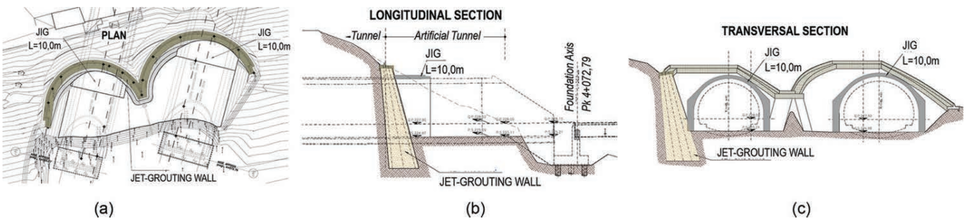


Figure 4.52 Portal jet-grouting shell: (a) plan, (b) longitudinal and (c) transversal sections (Lunardi et al., 2014).

Figure 4.53 Examples of portal of tunnels by jet-grouting shells. (a) S.Elia Tunnel, Messina-Palermo Motorway and (b) Tunnel N2, Sibari-Cosenza Railway Line (Lunardi, 2008).

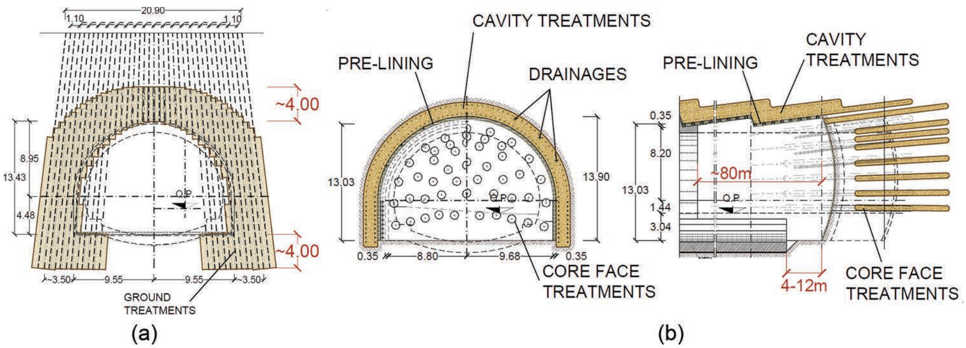


Figure 4.54 (a) Jet-grouting by surface. (b) Sub-horizontal jet-grouting treatment at the excavation face (Lunardi et al., 2014).

possible to overcome all the difficulties connected with excavation in cohesionless soils. Sub-horizontal jet-grouting is used to create a series of columns of improved ground, side by side ahead of the face around the profile of the extrados of the tunnel to be excavated. An arch of improved ground with considerable strength is created to provide protection to the ground inside the advance core along the longitudinal direction, lightening the load on it and giving it stability; this arch produces cavity confinement action occurring along the transverse direction, sufficient to prevent the ground around it from decompressing and consequent deformation from occurring (Figure 4.55). It therefore allows subsequent tunnelling operations to proceed under the protection of an arch effect already operational and therefore in complete safety. Less decompression and less deformation also mean less pressure on the final lining, which therefore needs to not be so thick. In addition, jet-grouting columns are used to stabilize the core-face (Lunardi, 2008).


(a) (b)

Figure 4.55 Jet-grouting arch for tunnel excavation (Lunardi et al., 2014) and core-face grouting (Lunardi, 2008).

4.6 ARTIFICIAL GROUND FREEZING

When temporary ground improvement in water-bearing soils is required for underground works, artificial ground freezing (AGF) may be a viable way to achieve it. With this technology, it is possible to create frozen ground bodies of appropriate thickness and characteristics to act as temporary soil support and/or waterproofing (Anderdson & Kinosita, 1978). Frozen ground behaves like a conglomerate where the binder function is accomplished by the frozen water (ice) filling the soil voids; it is impermeable, and its mechanical characteristics are remarkably improved compared to the natural soil. The technology is applicable to any type of soil, provided there is a sufficient amount of water/moisture (Harris, 1988).

In principle, AGF entails the installation within the soil of a pipework arrangement permitting the circulation of a cold medium within the volume of ground to be frozen; it is a system of freeze-pipes suitably spaced around the perimeter of the improved soil body to be built (Jessberger, 1980). The primary scope of AGF is to extract heat from the water-bearing soil until its temperature drops below the freezing point of the groundwater system (freezing phase). Each freeze-pipe forms a column of frozen soil; the columns grow and merge together with the adjacent ones, forming a resistant and impermeable retaining structure (Figure 4.56).

Then, the achieved temperature level is maintained by adjusting the flux of heat extracted from the soil (maintenance phase) until after the construction activities have been completed. Security is achieved once the final lining is in place, and from now on, the source of chilling can be switched off and natural thawing can commence (thawing can also be accelerated). Once thawing is complete, the soil conditions return to "normal" without interference or contamination of aquifers (Johansen & Frivik, 1980).

Freezing is achieved by circulating a coolant fluid through freeze-lances, made of two concentric pipes: the outer one has a closed end, while the inner one is open. Generally, the coolant fluid is pumped through the internal pipe down to its deepest point (Ladanyi & Sailes, 1978). On its way back through the annulus between the inner and outer pipes, the coolant fluid extracts the heat from the ground, thus decreasing its temperature (Jessberger & Vyalov, 1978).

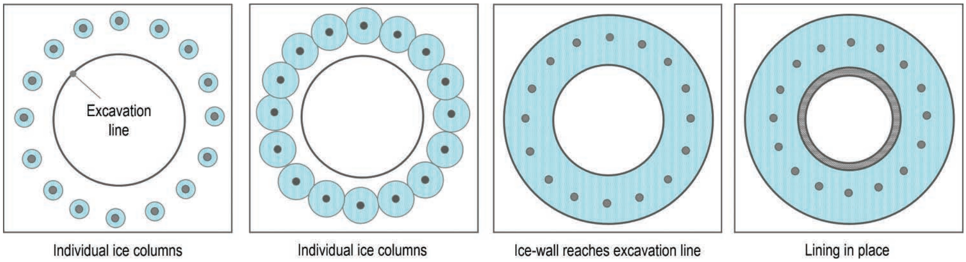


Figure 4.56 Stages in growth of an ice wall.

4.6.1 Freezing methods

Two different construction methods of AGF are available: indirect (with brine as coolant fluid) and direct (with liquid nitrogen as coolant fluid) methods. The indirect method (Figure 5.57) is a closed-circuit process that requires the use of an industrial refrigerating plant, connected to a coolant system, which consists of a brine pump, surface manifolds and freeze-lances installed into the ground. The brine, usually a calcium chloride ($CaCl_2$) solution, is cooled by the refrigerating plant, typically at temperatures of −28°C to −35°C, and pumped into the closed circuit. Premixed coolant fluids are now available on the market and may also be used as an alternative to brine. The warmer fluid returning from the freeze-lances through the insulated surface manifold system is then re-cooled by the refrigerating plant and re-circulated into the closed circuit. The indirect method relies on the pumped circulation of large flow rates

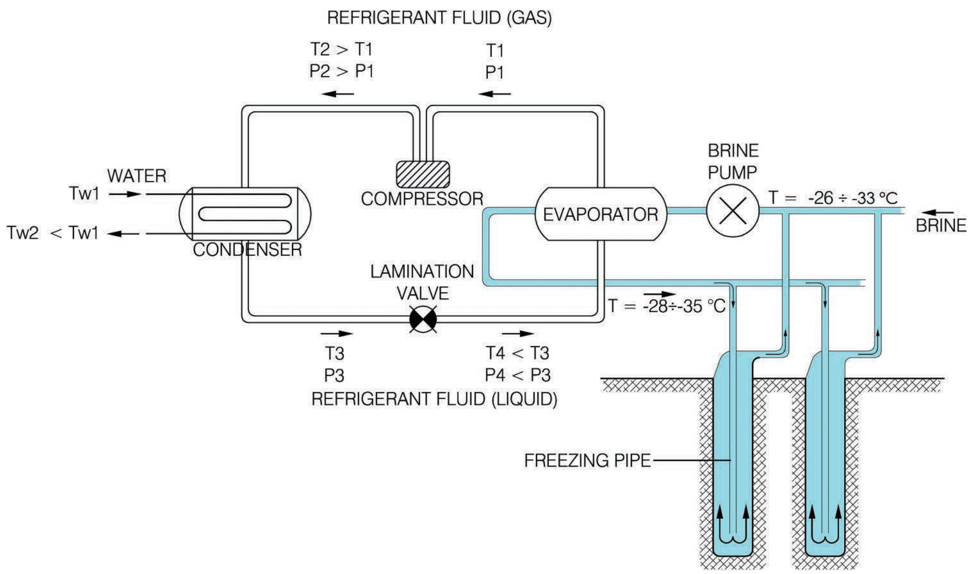


Figure 4.57 Indirect method. Brine cooled by a refrigerating plant, circulated into the freeze-pipes.

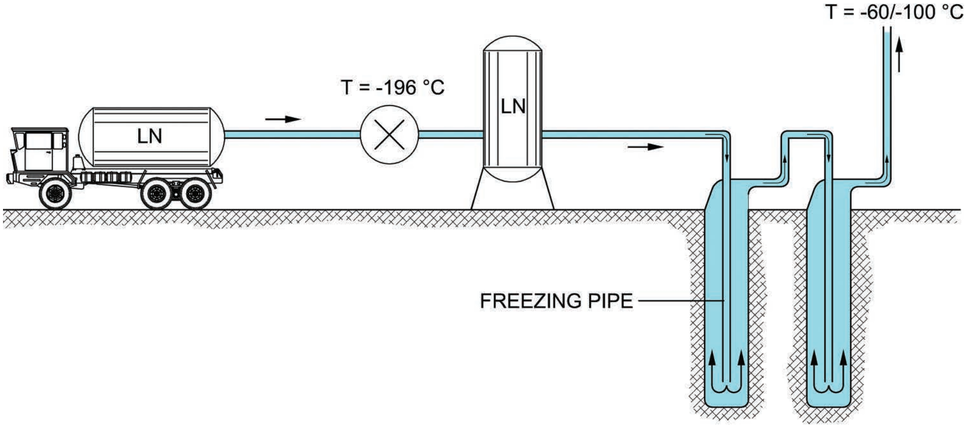


Figure 4.58 Direct system. Liquid nitrogen circulated directly into the freeze-pipes.

of coolant fluid and on a small temperature differential between delivery and return (Jones, 1980).

The direct method (Figure 4.58) is an open-circuit process. The coolant medium is liquid nitrogen (LN) at −196°C, an off-site-produced liquefied gas (cryogenic), delivered on-site in vacuum-insulated storage tanks. The LN is circulated directly into the freeze-pipes through an insulated manifold system. On its way along the freeze-pipes, the LN evaporates and extracts heat from the ground; the resultant gas is allowed to exhaust into the atmosphere at a temperature ranging from −100°C to −60°C. The direct method relies on a very high temperature differential between delivery and return, on relatively low flow rates of coolant fluid and on the thermal energy provided by the evaporation of LN. The direct method is quicker than the brine method in achieving the frozen body and attains a higher strength on the frozen ground, but it is generally more expensive.

The two methods can also be used in combination: as an example, the direct method may be applied to build up the design frozen body (freezing phase), and the indirect method, to maintain the achieved level of soil temperature (maintenance phase). But the indirect and direct methods may also alternate within the same freezing project or direct method adopted in localized areas where the build-up of the frozen body by brine is likely to be unsuccessful.

The choice of the freezing method is a function of numerous design, construction and economic factors, such as ground and groundwater condition, speed of underground seepage, required design strength and elasticity modulus of the frozen ground, planning, duration of the maintenance period, logistics and costs.

4.6.2 Freezing applications

Typical applications of AGF for underground works are both for shafts and for tunnelling. As far as the shafts are concerned, when there is the possibility for the freeze-pipes to reach an impermeable cut-off stratum, only a circle of freeze-pipes enclosing the shaft layout area is usually required (Figure 4.59a). If no impermeable cut-off stratum is present or reasonably attainable by the freeze-pipes, an appropriate bottom plug is required further to the basic freeze-pipe circle (Figure 4.59b). In principle, this bottom plug should be installed by a different technology other than AGF, because the possible central tubes necessary for a frozen bottom plug would interfere with the subsequent excavation of the shaft and require the anticipated switch off, thus entailing its premature thawing.

Shallow tunnels may be dealt from the surface when space permits, and surface structures, buried utilities or other obstructions do not interfere with the boreholes to install the freeze-pipes. Different possible solutions may be conceived by using either vertical or inclined freeze-pipes, depending on the presence or not of an impermeable cut-off stratum, as shown in Figure 4.60.

For tunnels where the installation of freeze-pipes from the surface is impractical or the overburden very deep, a horizontal or sub-horizontal arrangement of freeze-pipes may be selected. When the tunnel scope is to connect two underground spaces and the drilling length does not exceed 30–40 m, the arrangement shown in Figure 4.61 may be chosen. In case of drilling length exceeding the above-mentioned limit, this arrangement may be still adopted, but directional drilling has to be employed instead of conventional drilling.

In case the distance between the two underground spaces is not compatible with the drilling feasibility, a multiple horizontal stages procedure may be adopted (Figure 4.62), by installing the freeze-pipes in a slightly fan-shaped array to allow the excavation of an enlargement as the tunnel approaches the end of each frozen section; these can form

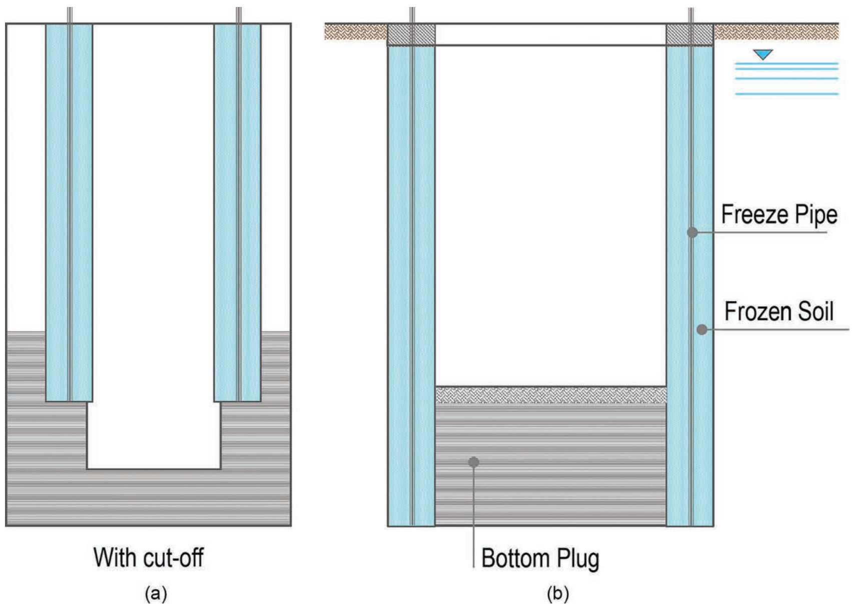


Figure 4.59 Freeze-pipe arrangement for shafts frozen from the ground surface: (a) with impermeable cut-off stratum; (b) without impermeable cut-off stratum.

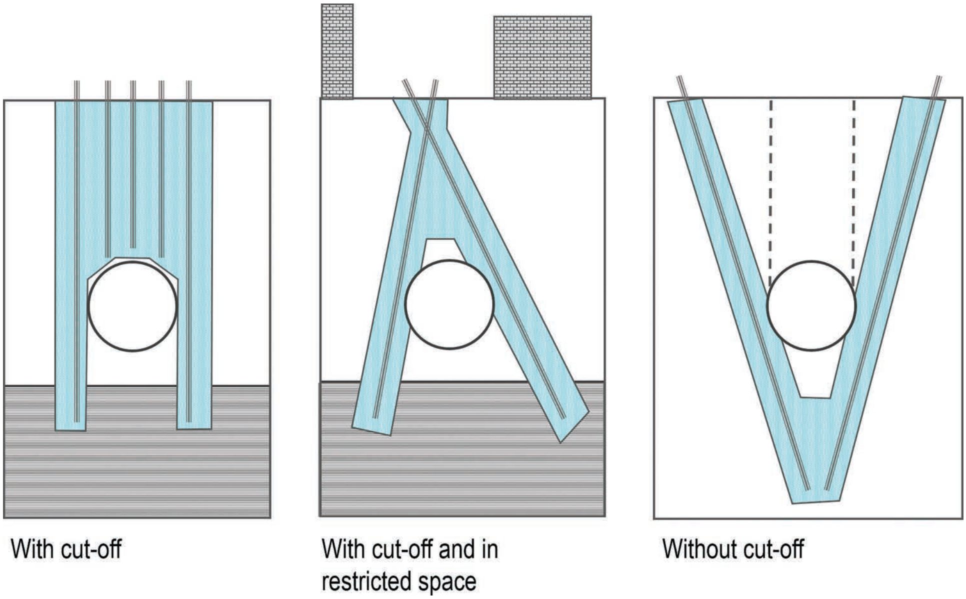


Figure 4.60 Freeze-pipe arrangement for shallow tunnels frozen from the ground surface: (a) with impermeable cut-off stratum; (b) with impermeable cut-off stratum and restricted surface space; (c) without impermeable cut-off stratum.

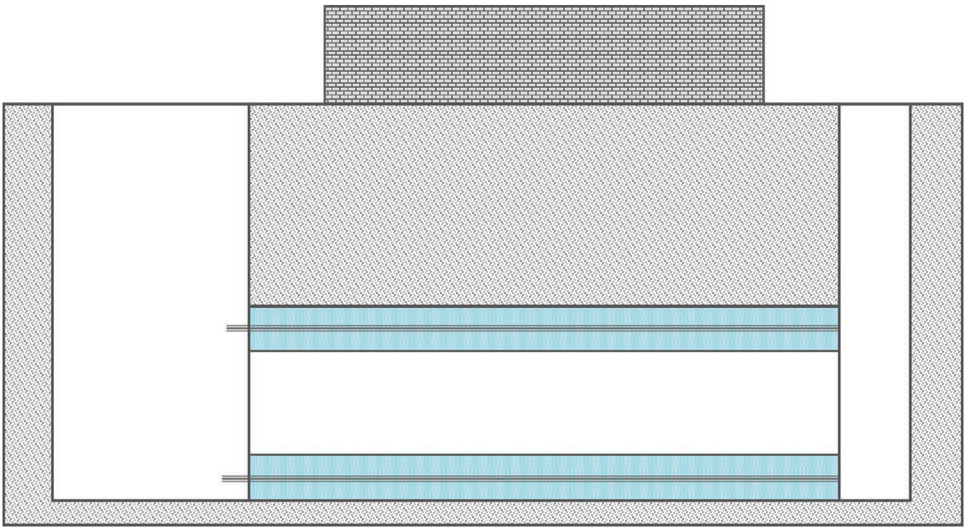


Figure 4.61 Freeze-pipe arrangement for a single-stage tunnel connecting two underground spaces.

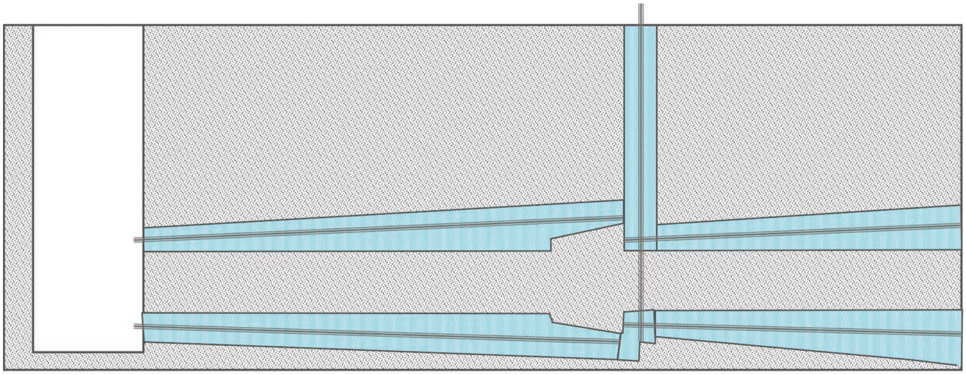


Figure 4.62 Freeze-pipe arrangement for a tunnel bored in multiple horizontal stages with frozen or grouted or jet-grouted bulkhead.

a drilling chamber from which further freeze-pipe arrays can be installed. In this case, an appropriate bulkhead is required at the end of each frozen section further to the basic freeze-pipe array. This bulkhead can be installed from the surface or from any existing parallel tunnel or underground space and may be achieved by AGF or grouting (installed from a level both above and below the water table) or jet-grouting (only from a level above the water table).

4.6.3 Monitoring artificial ground freezing

Monitoring is a crucial aspect for the AGF technology. Surveying the actual arrangement of freeze-pipes and temperature chains and monitoring of the soil temperature (within and around the volume to be frozen), coolant fluid temperature, groundwater pressures and displacements (on the existing structures and utilities nearby) are always to be implemented in AGF projects.

Monitoring should be always carried out by means of an automatic reading, acquisition and recording system. The result of monitoring should be shared in real time, through a specific Web platform, with all the key people involved in the project (owner, engineer, main and specialized contractor, consultant, etc.) in order to allow everybody to be up to date on the development of the frozen body and process as well as on possible groundwater overpressures and displacements, and to allow whoever may be concerned to react quickly if necessary (ISGF Working Group 1, 1991).

4.6.3.1 Surveying the actual arrangement of the net of freeze-pipes and thermometric chains

The ongoing survey of the actual arrangement of the net of freeze-pipes and thermometric chains is essential to preliminarily detect any potential weak point within the future frozen body, so as to be able to supplement with additional freeze-pipes, and also to understand temperature monitoring while freezing. The actual arrangement of freeze-pipes and thermometric chains may result significantly different from the anticipated theoretical design arrangement. This is due to both possible errors during the initial positioning of the drilling rig mast and unavoidable borehole deviations from the theoretical design axis. The longer the drilling, the greater are these deviations.

For this reason, each freezing borehole has to be surveyed and its actual path measured by suitable methods in order to gather the actual picture of the 3D arrangement of both freeze-pipes and thermometric chains. The actual paths are then included in an as-built 3D model in order to allow to calculate any distance between adjacent freeze-pipes and between thermometric chains and the closest freeze-pipe. The actual position of the freeze-pipes and thermometric pipes may also be drawn on several representative as-built cross sections.

When the borehole path survey shows any local spacing between adjacent freeze-pipes large enough to be potentially detrimental to the integrity of the frozen body to be built, additional boreholes have to be drilled and equipped with new freeze-pipes, in order to close these openings. The maximum allowed window opening has to be defined by the design technical specifications. These new boreholes have again to be surveyed for deviation in order to verify if the remedial target has been achieved. In case of failure in achieving the target, new additional boreholes are drilled and checked until around the whole surface of the body to be frozen there are no local zones with space larger than that fixed by the design technical specifications.

Furthermore, knowing the actual position of the thermometric pipes versus freeze-pipes allows the calculation of the distance between each temperature gauge equipping the thermometric chains and the closest freeze-pipe (i.e. the closest source of chilling). This information is a key factor for the understanding of the freezing process development and for the estimation of the temperature distribution within and surrounding the frozen body, during both the construction and the maintenance of the frozen body.

4.6.3.2 Temperature monitoring

Temperature monitoring during ground freezing entails the soil and the coolant fluid. Monitoring the soil temperature is necessary to gather information on and control the growth of the frozen body, while monitoring the fluid temperature allows to control and guide the freezing process. For this reason, a specific temperature gauge net is designed and installed within the soil nearby the freezing-pipes, within and around the design volume to be frozen, as well as at crucial points of the freezing circuit. Suitable temperature gauges are installed inside the purpose-drilled boreholes fitted with permanent casing-pipe. Chains of temperature gauges are assembled off-site, then inserted into the permanent casing-pipes and connected to a data logger for measurement, acquisition and recording of the temperature data. The longitudinal spacing of the gauges along the thermometric chains is to be specified by the design technical specifications (usually 1–3 m, as a function of the total length). Particular care should be taken in the choice of the type of gauges: their characteristics must be appropriate to the anticipated soil temperature during the whole freezing process, mainly depending on the type of coolant fluid selected (brine or LN).

Further temperature gauges are installed at the outlet and inlet points of the refrigerating plant(s) and at the inlet and outlet points of each freeze-pipe (or each series of freeze-pipes if a parallel connection is adopted). These gauges allow the estimation of the temperature differences between the entry to and the exit from the whole freezing circuit and each freeze-pipe (or series of freeze-pipes), thus allowing to appreciate the quantity of heat extracted by the net of freeze-pipes and by each freeze-pipe (or series of pipes), depending also on the fluid coolant flow rate.

The interpretation of the soil temperatures during freezing cannot avoid correlating the measured temperatures with the distance between each temperature gauge and the closest freeze-pipe. In fact, the temperature measured by each gauge is obviously a function of its distance from the closest chilling source, following the rule that a closer chilling source entails a colder soil and a more distant chilling source entails a warmer soil.

Knowing the actual position of the thermometric pipes vs. freeze-pipes is furthermore of great help when estimating the temperature distribution within the frozen body during freezing (both freezing and maintenance phases) and thus for understanding the development of the freezing process. Plotting the measured temperatures versus the distance between each gauge and the closest freeze-pipe, we can find a correlation curve at any moment of the freezing process, thus allowing to estimate at any time the position of the different isotherms as well as the evolution of the frozen front. The actual temperature distribution within the frozen body during freezing is well represented by a logarithmic curve, as suggested by the theoretical approaches proposed by different authors (among others, Sanger & Sayles, 1979) and confirmed by several freezing job site experiences.

4.6.3.3 Water pressure monitoring

When ground freezing is designed to form, together with other elements, a closed and impermeable chamber, a monitoring of the water pressure inside the chamber is mandatory. Monitoring is usually performed by means of one or more open pipe piezometers installed within the unfrozen core to be excavated, fitted with valve and pressure gauge. Water pressure monitoring has two purposes:

a. to detect the moment of the “closure” of the frozen shell by a sudden increase in the water pressure;
b. to detect any unsuitable overpressures on any of the structures forming the boundary of the closed chamber and, if necessary, release the water pressure by opening the valve and letting the piezometer pipe act as a drain pipe.

Furthermore, once the shell has achieved its “closure” and the design thickness, and before starting excavation or tunnelling, the piezometer pipe may be used as drain pipe, by just opening the valve, to let the water to flow out, thus providing drainage for the unfrozen core to be excavated.

4.6.3.4 Displacement monitoring

The scope of monitoring is to survey the effects of soil freezing on the existing nearby structures and utilities. Displacements may occur both during freezing and during thawing (once the freezing has accomplished its scope). In the first case, a frost heave or horizontal displacement may be expected, but if the movement is restrained, for example by a building load or by a retaining structure, a significant frost pressure may develop. Conversely, in the second case a thaw settlement is expected.

The frost heave/horizontal displacement is a complex phenomenon related to the expansion of soil while freezing. When the temperature is lowered to the freezing point of the pore water, water in soil pores becomes partly frozen and expands by 9% of its unfrozen volume. In fine-grained, frost-susceptible soils, the induced suction force causes additional water migration to and in the freezing zone. On freezing, the migrated water forms ice lenses and causes volumetric expansion along a direction perpendicular to the freezing front. In coarse-grained soils, non-frost-susceptible soils expansion is supposed to be induced by closed-system freezing only and in cases when the pore water can flow out of the freezing zone, no expansion is expected (ISGF Working Group 2, 1991).

Thaw settlements are the generally uneven downward movements of the ground surface due to thaw consolidation (Harris, 1995). It has different possible components, namely volume reduction due to phase change, self-weight of the soil and de-structuration of the soil while freezing.

Displacement monitoring may therefore be very useful and allow for suitable action to be taken to counteract frost heave/horizontal displacements (by properly controlling the freezing process during both construction and maintenance of the frozen body) and thaw settlement (by, for example, compensation grouting).

4.6.4 Summary

AGF is a technology for temporary soil improvement in water-bearing soils, selected to create frozen ground bodies of appropriate thickness and characteristics to act as temporary soil support and/or waterproofing. When the scope of AGF is to allow a safe and almost dry excavation or tunnelling below the water table, the design of frozen body contributes, together with other elements (such as starting and arrival shafts, and natural or artificial bottom plugs or bulkheads), to forming the boundary of a closed and impermeable chamber, which will protect the excavation from collapse or intrusion of groundwater and loose soil. Excavation or tunnelling is then carried out within this closed chamber.

From the design point of view, although geotechnical design is the main aspect in AGF, it should be accompanied by a thermal analysis, in order to obtain the temperature distribution, elapsed freezing time and energy required to perform the job (Frivik, 1980), and also by the technological design. From the construction point of view, further to excellent engineering and a very skilled and experienced geotechnical contractor, quality control and real-time monitoring are key factors for a successful ground freezing job. Surveying of the freezing boreholes and real-time monitoring of temperatures, groundwater pressure and displacements are of utmost importance to guide the freezing process well and to guarantee the best final result.

4.7 PRECUT AND PRETUNNEL

In order to modify the rigidity of the advance core and to thereby create the right conditions for the complete control of the deformation response of the stiffness of the ground under bad stress-strain conditions, protective intervention is required. The channelling of stresses around the advance core performs a protective function that ensures that the natural strength and deformation properties of the core are maintained. Interventions such as mechanical precutting or pretunnelling, which treat the future perimeter of the cavity, are carried out to assure cavity preconfinement (Lunardi, 2008).

4.7.1 Cavity preconfinement by means of full-face mechanical precutting

The development of mechanical precutting technology is based on the idea to make a cut around the outer profile of the future tunnel and to immediately fill it with shotcrete so that a "primary lining shell" with good strength properties is rapidly formed (Arsena et al., 1991).

Full-face mechanical precutting consists in making an incision of a predetermined thickness and length around the line of the extrados of the future tunnel (Figure 4.63). The incision is made by using a special machine equipped with a chain cutter (Figure 4.64) moving on a rack and pinion portal reproducing the shape of the tunnel outline and is immediately filled with fibre-reinforced sprayed concrete with appropriate additives to give it rapid strength (Figure 4.65). A pre-lining tile is thereby created with a truncated cone shape and good mechanical characteristics, which projects well ahead of the face to provide radial preconfinement of the surrounding ground sufficient to prevent the rock mass around it from loosening (Figure 4.66).

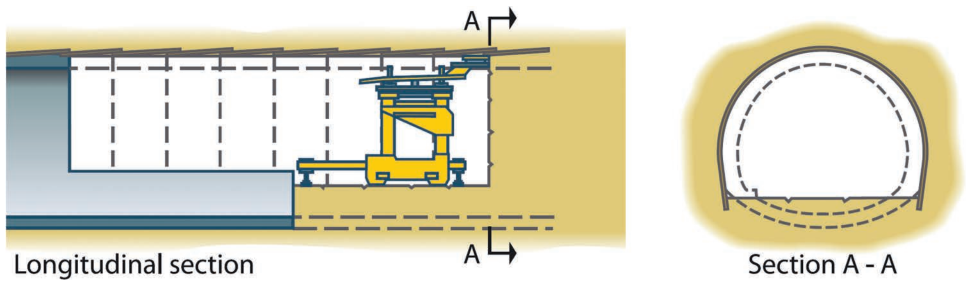


Figure 4.63 Full-face mechanical precutting (Lunardi, 2008).

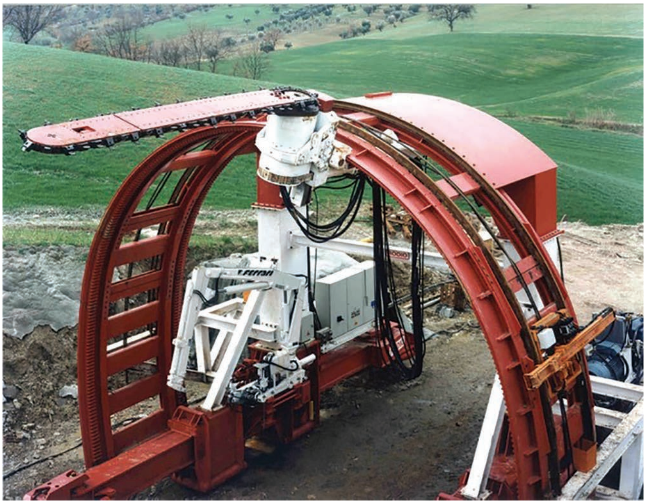


Figure 4.64 Precutting machine (Lunardi, 2008).

Figure 4.65 Cavity preconfinement by means of full-face mechanical precutting: execution and filling of the precut (Lunardi, 2008).

Important characteristics of the method are the following:

- the almost total elimination of overbreak between the preliminary lining and the ground;
- the very high degree of mechanization that provides regular advance rates.

Figure 4.66 Cavity preconfinement by means of full-face mechanical precutting: view of the core-face protected by a mechanically precut shell (Sibari-Cosenza Railway Line; Lunardi, 2008).

Good stability of the core-face is determinant for the success of the system. If the protection provided by the precut shell is not sufficient to guarantee this, then it is essential to reinforce the advance core by placing, for example, fibre glass reinforcing elements (Figure 4.67). The operational stages for full-face, mechanical precut, construction are illustrated schematically in Figure 4.68, which illustrates the very common case where this technology is applied in combination with reinforcement of the core-face. Under the water table, it is important to con-serve the integrity of the core-face by the systematic creation of a fan of drainage pipes ahead of the face, always launched from outside the core-face (Lunardi et al., 1994).

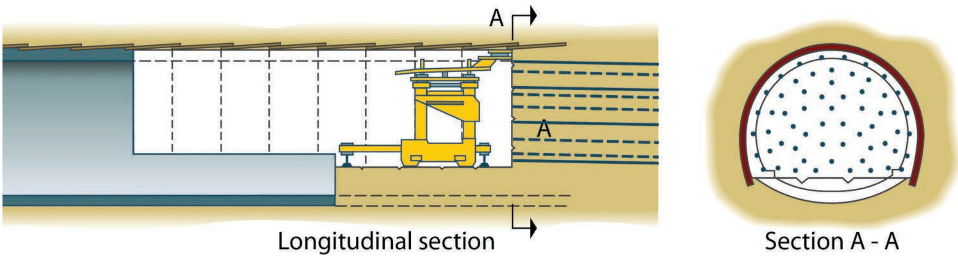


Figure 4.67 Cavity preconfinement by means of full-face mechanical precutting and reinforcement of the core using fibreglass element (Lunardi, 2008).

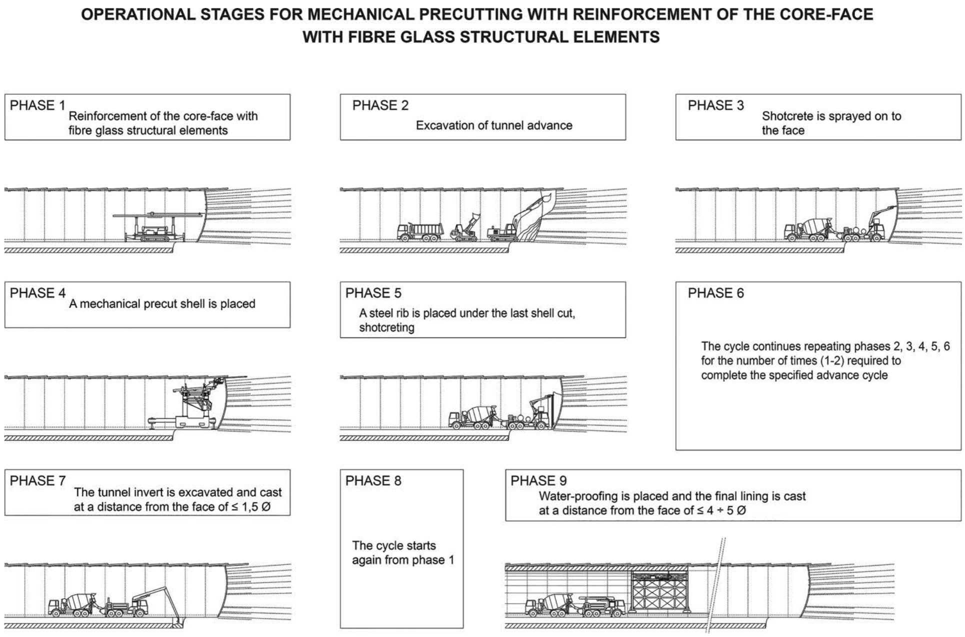


Figure 4.68 Operational stages for mechanical precutting with reinforcement of the core-face with fibreglass reinforcing elements (Lunardi, 2008).

4.7.2 Cavity preconfinement using pretunnel technology

The pretunnel technology was developed in Italy during the 1990s (Peila et al., 1995). The technology gives the possibility of constructing the tunnel lining before it is excavated, and it involves the creation of a truncated cone of concrete in advance ahead of the face, which can become an integral structural part of the final tunnel lining or even replace it altogether, depending on the thickness adopted and the design decisions (Gattinoni et al., 2014).

Pretunnel technology basically differs from mechanical precutting for:

- tile thickness (40–80 cm instead of 18–24 cm) and length (8–10 m instead of 3–4 m);
- the use of concrete instead of shotcrete or mortar as a filler.

Pretunnel technology is strictly dependent on the equipment designed especially for it. These consist of a self-propelled tubular frame fitted with an arm held by rotating telescopic locking arms powered by six hydraulic motors (Figure 4.69). The cutter module consists of two chains, assembled on a single rigid arm with a box structure. The chains are fitted with discs and spikes, and the geometry, cutting angle and number of cutters can all be varied according to the nature of the ground. Finally, a slip form for casting concrete is fitted behind the cutter module. It moves as a fixed piece together with the module and concrete is pumped into it through special tremie pipes to fill the

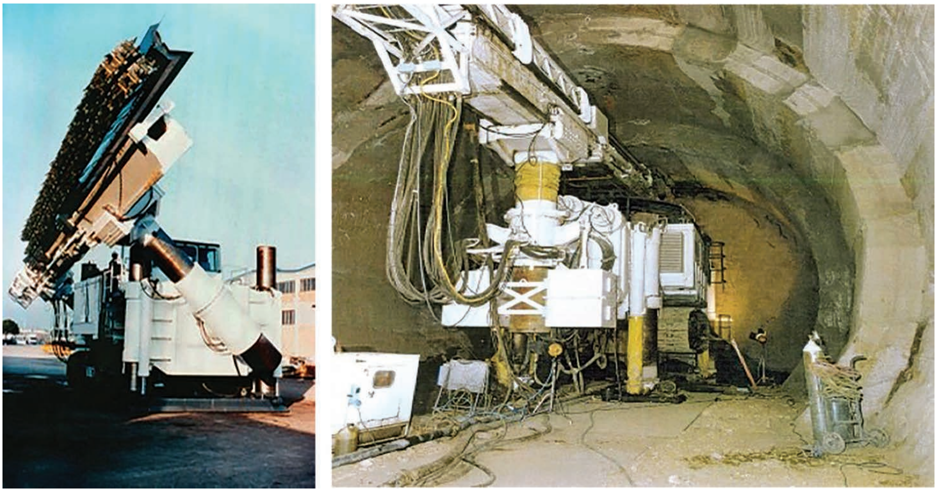


Figure 4.69 Pretunnel equipment (Lunardi, 2008).

excavated area (Lunardi et al., 1997). Figure 4.70 shows the pretunnel advance system (Tonon et al., 2005).

4.7.3 Precutting: the evolution for tunnel widening

As traffic increases, there is an ever greater need to widen roads, motorways or railways to increase their capacity. Meeting these needs is very complicated when they run through stretches of tunnel.

A tunnel can be widened if it is possible to:

- Guarantee the necessary safety of users and limit inconvenience below an acceptable threshold.
- Solve both the technical and operational issues connected with driving the face to widen the tunnel in ground that has already been disturbed by a previous excavation.
- Construct the new load-bearing structure at the same time as the old one is demolished and deal adequately with any stress-strain conditions, even unexpected, which might be met during construction, without any danger to tunnel users and to any human activity there on the surface.

This widening process reduces the impacts on both the environment and infrastructural management, but a highly mechanized and industrialized construction method must be used to guarantee not only the safety of workers and users, but also a time of construction not impacted by the presence of traffic. The right solution has been settled out by adapting the characteristics of the machines and equipment developed for precutting and pretunnelling (Lunardi, 2008).

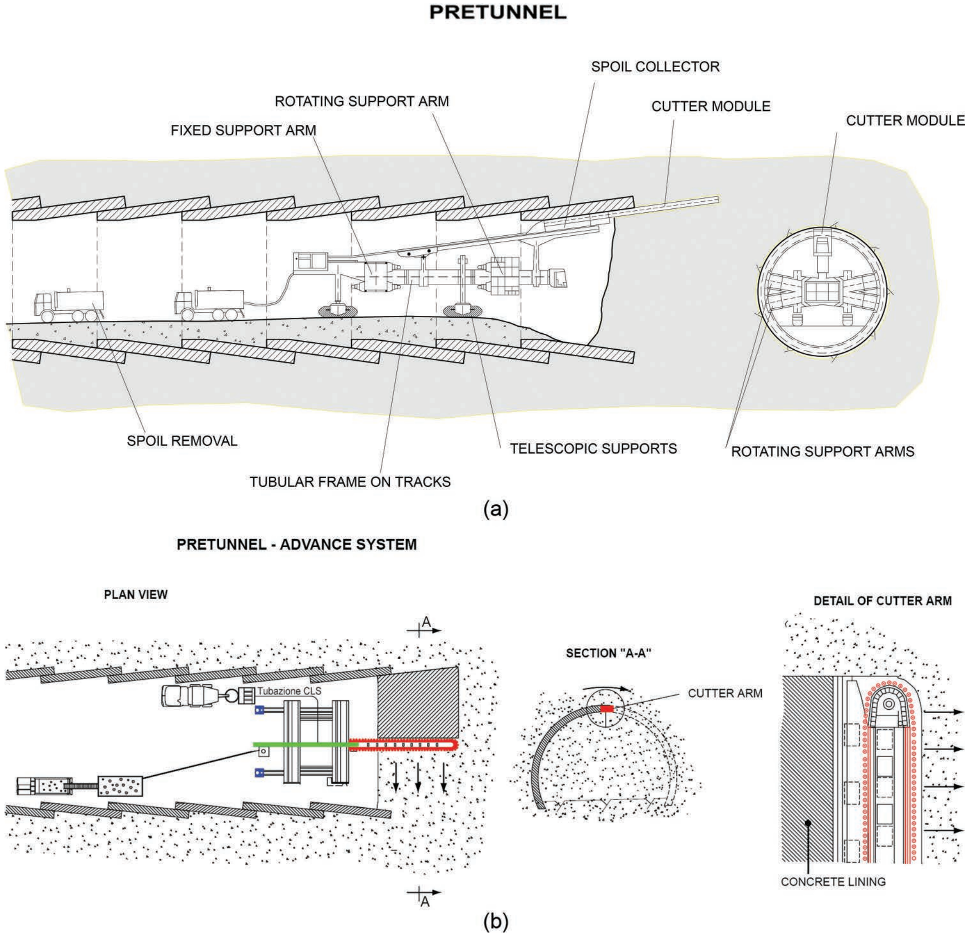


Figure 4.70 Pretunnel advance system: (a) transversal view; (b) plan view.

The widening of a tunnel in service without interrupting traffic requires to solve two main problems:

- to excavate and construct the lining of the widened tunnel, to demolish the existing tunnel, while ensuring the safety of tunnel users and minimizing inconvenience;
- to adapt the technique for use in any type of ground and stress-strain condition it might encounter.

The construction cycle adopted for the widening is as follows: a pre-consolidation of the vault by mechanical precutting and a sequence of various cycles of excavation and segmental lining installation, more modernly declined by precast segments installation

(Lunardi et al., 2016a). In addition, an element with appropriate structural properties, called "traffic protection shield", guarantees safety for traffic and construction workers by separating the working areas from the road. In order to perform the pre-consolidation of the vault with precutting technology and the active vault lining, a special machinery has been specifically designed and manufactured with an arched shape structure capable of operating over the traffic shield protecting the motorway below (Figure 4.71a). The machinery is composed by:

- Dedicated equipment for precutting and backfilling, with a control cabin and mobile consoles for a better control of all the operations (Figure 4.71b).
- Robotic arm for the assembly of the segmental lining and backfilling of the extrados, with fixed and mobile consoles for control and assistance during all the operations (Figure 4.71c).

The traffic protection shield (Figure 4.71d) has to comply with the following safety requirements:

- Resistance to the impact with any material which could fall from above.
- Appropriate dimensions to allow the transit of two lanes below the shield.
- Resistance to possible impacts from vehicles.

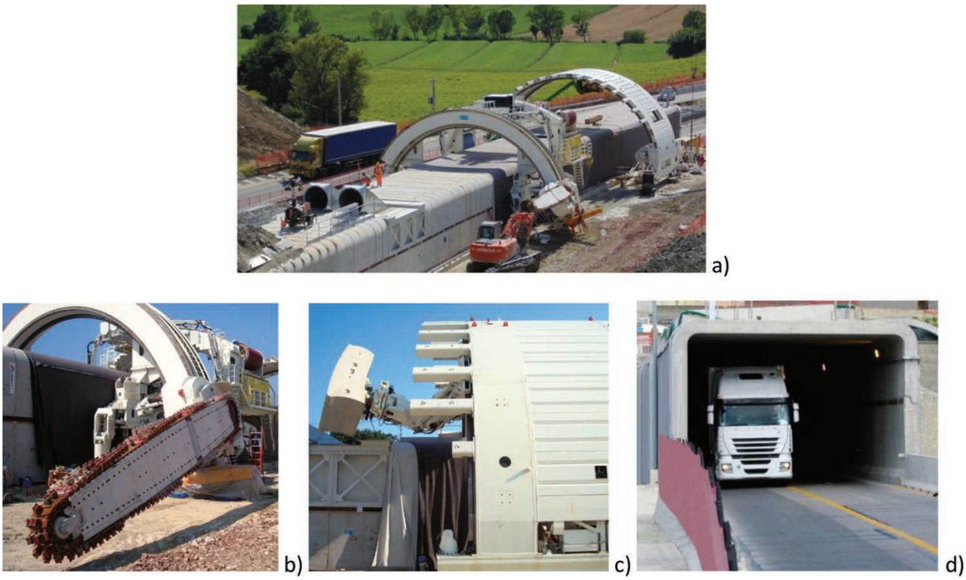


Figure 4.71 (a) Equipment for precutting and segmental lining installation operating over the traffic protection shield; (b) precutting tool; (c) robotic arm; (d) traffic protection shield.

4.7.3.1 Working cycle

Three main phases characterize the working cycle.

Mechanical precutting consists in an incision on the excavation face at the extrados profile of the tunnel to be widened with length and thickness relevant to the geotechnical characteristics of the ground and the backfilling of the incision itself with fibre-reinforced concrete in order to form a shell that resists the pressures of the surrounding mass (Figure 4.72a and b).

The excavation of the widened section and demolition of the existing tunnel are carried out in steps of 2 to 4m, depending on the mechanical characteristics of the mass affected. All works required for the advance of the excavation are conducted simultaneously on both sides of the shield. The dimensions and characteristics of the equipment for excavation and spoil removal are selected to operate in the confined spaces between the lining and the protection shield, while maintaining the ability to operate at a significant height, which can be in excess of 10 m. Spoil removal and shotcreting of the face are performed by suitable equipment using the upper level of the extrados of the traffic protection shield as working platform (Figure 4.72c and d).

The final lining is formed by precast segments placed according to a very short distance from the face. The installation of the lower side segments can be carried out simultaneously on both sides of the tunnel with the use of crane trucks, while the segments at the crown are installed with a specific operating machine able to support the weight of the arch up to the expansion of the arch itself by means of flat jacks at the key segment, which make the vault self-supporting. The void between the precutting and the extrados is filled by shotcrete (Figure 4.72e and f).

Because of the reduced dimensions of the tunnel within the shield, in order to prevent the access in the tunnel of oversized vehicles that could impact on the shield causing serious risks to the traffic, some measures must be adopted for traffic management on-site. In addition, fire protection and emergency surveillance systems have to be implemented (Lunardi et al., 2016).

The widening method can be applied in both cohesive and granular soils. The traffic reduction shield represents an operational opportunity and constitutes the main and safest route to work on the widening face. In addition, this construction procedure finds its natural field for development where no alternatives are possible as in the case of either urban road or rail tunnels with exits onto viaducts or other works of art.

4.7.4 Summary

In order to regulate the stiffness of the advance core and to thereby create the right conditions for the complete control of the deformation response of the ground, protective interventions are required. The protective interventions channel stresses around the advance core and ensure that the natural strength and deformation properties of the core are conserved. Intervention treating the perimeter, such as mechanical precutting or pretunnelling, are carried out to perform cavity preconfinement.

Full-face mechanical precutting consists of making an incision of a predetermined thickness and length around the line of the extrados of a future tunnel. The incision is made by using a special machine equipped with a chain cutter, which moves on a rack

Figure 4.72 Tunnel widening working cycle: (a) precutting execution; (b) precutting incision; (c) widening section demolition; (d) widening section excavation; (e) segmental lining installation; (f) void filling between the segments' extrados and the precutting shell.

and pinion portal that reproduces the shape of the tunnel outline and is immediately filled with fibre-reinforced sprayed concrete with appropriate additives to give it rapid strength. A pre-lining tile is created with a truncated cone shape projecting well ahead of the face, to provide radial preconfinement to the surrounding ground, sufficient to prevent the rock mass around it from loosening.

Pretunnelling involves laying of the final load-bearing structure before excavation, which consists merely of removing the earth enclosed within the tunnel lining which is already functional.

Precutting technology can be successfully applied in the case of tunnel widening.

4.8 DRAINAGE

The control and regulation of the infiltration of water plays a primary role in the stability of an underground excavation and also in mitigating its impact on the surrounding environment. A tunnel advancing inside an aquifer with no intervention to waterproof it has the effect of a huge drain: a natural infiltration movement is created along the excavation direction which, if not adequately dealt, can in some situations lead rapidly to the collapse of the core-face and of the cavity.

The water has a harmful effect on strength and deformability characteristics of both rocks and soils. This effect, which is the result of the reduction in the effective cohesion and friction values, can become particularly negative under hydrodynamic conditions when the fine fractions cementing the fissures in rock are washed away. As a consequence, whenever a tunnel intersects an aquifer, it is extremely important for the design engineer to assess the effects of the presence of water on the stress-strain state of the rock mass around the future tunnel (Figure 4.73) and to take the action required to ensure the short- and long-term stability of the cavity (Lunardi, 2008).

The designer must decide on the basis of (a) the water table recharge flow rate, (b) the hydraulic gradient, (c) the geomechanical characteristics, (d) the primary and secondary permeability of the rock mass and (e) the radius of the tunnel:

- whether to reduce artificially the permeability in order to perform excavation under hydrostatic conditions;

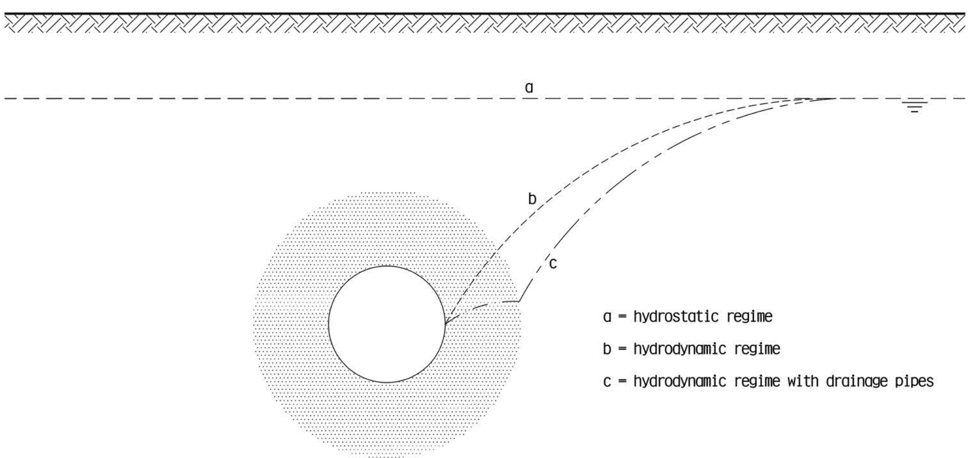


Figure 4.73 Tunnel excavation intersecting an aquifer.

- whether, on the contrary, to decide to advance under hydrodynamic conditions by channelling the entrainment forces caused by the movement of the fluid away from the perimeter of the excavation by means of drainage;
- finally, whether to employ combined waterproofing and drainage intervention.

In making that decision, the design engineer must consider that a hydrostatic regime is unacceptable when the hydraulic load is very high; on the other hand, to advance under hydrodynamic conditions may not be tolerated for environmental impact reasons if the rate at which the water table recharges is not sufficiently fast.

If, however, he has decided to allow drainage by the cavity, the hydraulic regime will pass through a transitory period during which the hydraulic pressure will fall sharply as a result of water filtering through towards the walls of the tunnel.

In the absence of adequate countermeasures, these phenomena affect the advance core first, with an appreciable reduction in its geomechanical strength and therefore also in its rigidity. Since the rigidity of the advance core plays a determining role in the long- and short-term stability of a tunnel, it is important for water to be prevented from circulating inside it. This can be achieved by intercepting the water three or four tunnel diameters ahead of the face with special drainage pipes arranged in an umbrella configuration around the future tunnel (protection of the advance core, Figure 4.74). It is very important for the design engineer to give very precise specifications to ensure that the drainage pipes are constructed exactly as required in order to prevent drainage ahead of the face from being ineffective because implemented wrongly with effects that may be the opposite of those desired.

More specifically, it must be absolutely forbidden to insert them in the ground from the surface of the face. They must be arranged in a truncated cone configuration and must start from the side walls (Figure 4.75) of the tunnel or at the most from the perimeter of the face so that the core is never intersected. Alternatively, the water

Figure 4.74 Cavity preconfinement by means of truncated cone "umbrellas" of drainage pipes ahead of the face (Sibari-Cosenza Railway Line, tunnel 4, 1998, ground: silty sand, overburden: ~ 40).

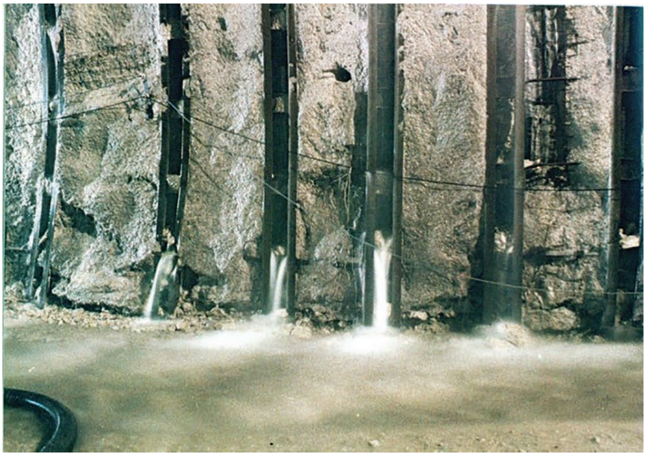


Figure 4.75 Cavity preconfinement by means of truncated cone "umbrellas" of drainage pipes ahead of the face: water drainage through NP jointed steel ribs (Gran Sasso Motorway Tunnel).

drawn in by them will soak the ground forming the core with disastrous effects for its stability and also therefore for the stability of the cavity. In order to prevent this danger, it is also important to specify that the drainage pipes must also have no perforations in them for a length of a few metres from the end closest to the tunnel (Lunardi et al., 2016).

Similarly, and for the same reason, the designer must provide clear instructions for the correct execution of all the reinforcement treatments involving drilling and then inserting reinforcement structures. It is important to drill one hole at a time and for it to be filled immediately and sealed perfectly with mortar. This is the only way to prevent it from rapidly becoming a channel for water to flow through with devastating consequences for the advance core, which once soaked and softened, would no longer be able to perform its stabilizing action effectively.

The design of the final lining also requires some extra attention. It should be protected from the potentially aggressive action of water by placing a waterproof sheet and a protective geotextile layer. If the water table is eliminated permanently, then the designer must also specify the placement of channels at the foot of the sidewalls or under the tunnel invert to run drainage water away. However, if it has been decided to restore the natural piezometric level when construction is complete, then the waterproofing must be sufficient to make the tunnel inactive hydraulically. There will then be a second transition regime ending when the original hydraulic head is restored.

4.8.1 Cavity preconfinement by means of truncated cone "umbrellas" of drainage pipes ahead of the face

The implementation of truncated cone "umbrellas" of drainage pipes under the water table, when the designer considers as the best choice to advance under hydrodynamic

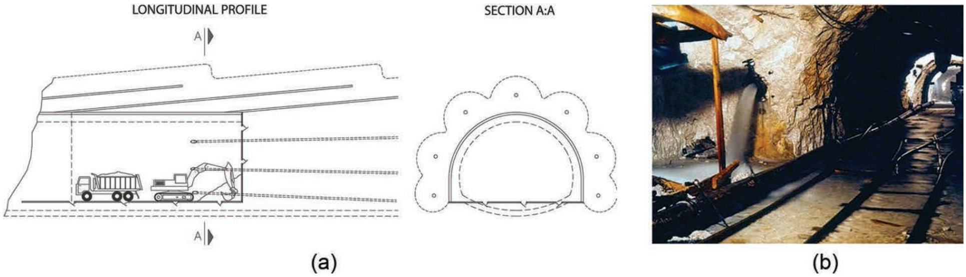


Figure 4.76 (a) Cavity preconfinement by means of truncated cone "umbrellas" of drainage pipes ahead of the face; (b) Wall drain (Gran Sasso Motorway Tunnel).

conditions, creates a zone of high permeability around the cavity, which will partially or fully decrease the level of the water table and also, as a consequence, that of the hydraulic gradient.

The hydraulic pressure on linings is also reduced appreciably. Since from a geomechanical viewpoint, the shear strength of the material is negatively affected by the presence of neutral pressures inside the pores and discontinuities of the ground, lowering these neutral pressures will improve the shear strength considerably.

In order for this improvement to be effective for the band of ground around the tunnel and for all the ground within the advance core, these drainage pipes must be launched ahead of the face according to a truncated cone "umbrella" configuration and they must always be positioned outside the core-face. This is the only way to prevent entrainment effects from being produced in the core resulting from the seepage flow which would damage its stability (Figure 4.76).

4.8.1.1 Operational stage

Truncated cone "umbrellas" of slotted drainage pipes are launched ahead of the face to intercept the flow of water in the ground around the advance core of the tunnel, to prevent the circulation of water inside the core-face and to reduce the hydraulic pressure. This improves the natural strength and deformability characteristics of the advance core, enabling it to exert an appreciable cavity preconfinement action. To achieve these goals, drainage pipes have to be positioned strictly outside the core-face, on the extrados of the perimeter of the cross section to be excavated with a truncated cone geometry, which is repeated in sequence when tunnel advance resumes so that the core is always completely protected. In practice, the stages of tunnel advance and the placing of drainage pipes must always be performed in an alternating sequence so that a continuous succession of overlapping truncated cone "umbrellas" is obtained with a length of not less than the diameter of the tunnel (Lunardi, 2008).

The length of the drainage pipes used will depend on the diameter of the tunnel, the permeability of the ground and the characteristics of the water table through which the tunnel passes. It will also be influenced by the available operating machinery. The pipe length is generally around three times the diameter of the tunnel.

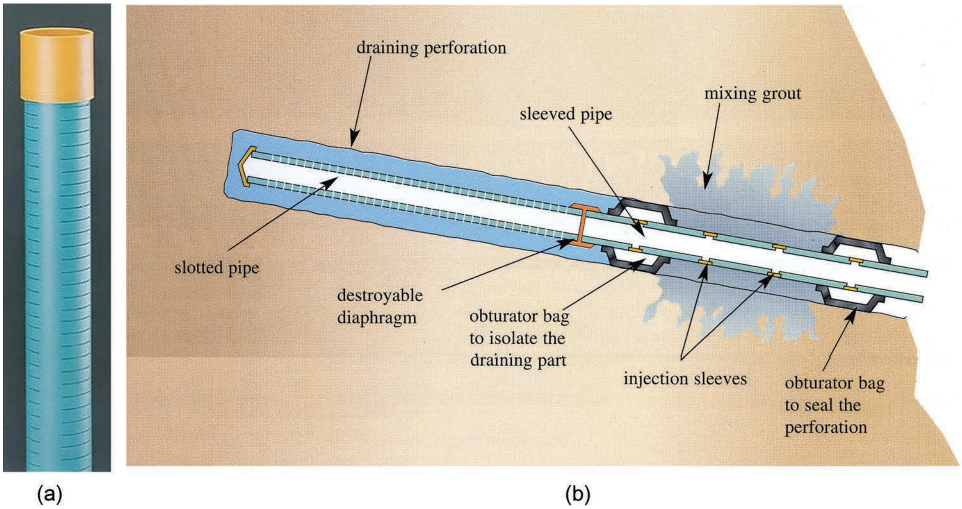


Figure 4.77 (a) PVC drain; (b) installation.

They are placed while tunnel advance is halted after first drilling the borehole, with destruction of the core, in which the drainage pipe is inserted. The pipe is usually in PVC with an initial slotted section (bottom of the borehole, to capture water) (Figure 4.77a), covered with a sleeve of geotextile (non-woven) to prevent clogging, and a second non-perforated section, which is cemented in with the walls of the borehole by injecting grout (Figure 4.77b). The two sections are separated externally by a membrane preventing the drainage water from hindering the injection operations. An obturated bag positioned outside the tube and filled with cement grout through a valve specially positioned on the wall of the pipe immediately ahead of the membrane isolates the perforated section from the non-perforated section when the grout injections are performed (Lunardi, 2008).

From an operational viewpoint, the installation of sub-horizontal drainage pipes is performed according to the following stages.

1. Boreholes are drilled according to the geometry specified in the design after first protecting the wall of the face with a layer of shotcrete. The drilling methods used will depend on the nature of the material treated and on the local hydrogeological conditions.

   The equipment should, however, satisfy the following requirements:
   - possibility of drilling holes by means of temporary lining at least 15 m long, without the need of manoeuvring the rods, keeping faithfully to the truncated cone geometry as specified at the design stage;
   - a rotary head with through hole and not too bulky in terms of distance between extrados and bore axis;
   - a sufficiently rigid design of the slide carriage, the guide devices for the rods and the positioning equipment in order to guarantee drilling to be performed within the specified geometrical tolerances.

Drilling must be performed using tools appropriate for the design diameter and allowing the subsequent operations for inserting the slotted pipes without any problem. If the walls of the boreholes are unable to support themselves long enough for the drainage pipes to be inserted, then drilling sessions must be interrupted at intervals for the insertion of a temporary lining, which is left in place until the drainage pipe is inserted. In this case, when the flow rate and the pressure of the water are high, the mouth of the borehole is tightly sealed by using a pipe fitting containing a stuffing box (which is connected using a waterproof joint to the drill tube) placed on the face at the axis of each borehole and an external discharge outlet which will allow the controlled outflow of the cuttings and prevent entrainment phenomena from being triggered. If necessary, this pipe fitting, which is known as a "preventer", allows the borehole to be closed rapidly if uncontrolled flows of water and/or particles of soil occur. It is recommended that the preventer be set rigidly into the surrounding ground with appropriate cementation.

2. Insertion of the drainage pipe with the borehole enclosed into a geotextile sleeve to protect the slots from clogging and activation. If the walls of the borehole are stable, then the micro-slotted tube can be simply caulked and externally sealed after insertion, while the water that is captured must be carried in pipes away from the tunnel face. Otherwise, a temporary lining must be inserted, especially if working under high pressures, which involves the following operations:
   - insertion of the slotted tube inside the temporary lining;
   - retrieval of a length of the borehole lining corresponding to the length of the slotted section of the drainage pipe and the packer, so that the latter comes in direct contact with the walls of the borehole;
   - the packer bag is filled with cement mixture under controlled pressure, injected through a special valve. Once the sack has been filled, the injection pressure is maintained until the mortar mixture in the packer has set, at which point the lining tube is withdrawn completely and grout is immediately injected into the non-perforated part of the drainage pipe starting with the most distant valve;
   - the internal membrane located next to the packer is broken using a normal rigid rod inserted into the drainage pipe.

   Depending on the type of ground and the likelihood of the slots in the drainage pipe becoming clogged, it may be necessary to unblock the pipe by pumping water into it at a constant rate and at a pressure greater than the external hydrostatic pressure for the time needed to clear completely the drainage section.
3. Inspection of the functioning of the drain. This inspection must be performed immediately after the drainage pipe is placed and then periodically for the whole of its operational life. The function performed by drainage pipes with respect to intervention to stabilize tunnels is often fundamental to the safety of a tunnel in both the long and short terms. The risk of a drainage pipe clogging must never be underestimated since it can occur as a result of countless different causes, even after a very long time.

   The geotextile filter plays a very important role, and it must be chosen very carefully on the basis of the local characteristics of the ground. It is then placed with appropriate precautionary measures taken to avoid damaging it and to allow it to perform its function as well as possible.

Finally, it is important to channel drainage water appropriately to prevent it from spreading in the tunnel where it might compromise subsequent stabilization operations (Lunardi et al., 2016).

4.8.2 Particular cases

In some particular cases, such as the one of greatly weathered conditions of the phyllitic schist in relevant presence of clay, associated with high pore water pressures, the amount of drainages installed at the boundary of the tunnel not adequately reduced the pore water pressure. In this case, it could be useful to install at the face some special reinforcing elements combining the reinforcing function with a drainage action. The special soil nailing element is made of a coaxial connection between a reinforcing component and a drain. Referring to the reinforcement section the presence of an expandable geotextile sheath wrapping the fibreglass reinforcing pipe for the whole of its length and is sealed at both head and tip. The drain is a micro-slotted HDPE pipe protected by a non-woven fabric geotextile (Sterpi et al., 2013; Figure 4.78).

Once the nail is inserted in a previously drilled borehole, a low shrinkage cement-based grout is injected through a small tube between the pipe and the sheath, so that the sheath inflates till the gap in the borehole is closed. The water collected from the deepest portion of the nail is drained out within the corrugated pipe, driven by natural on-site pore pressures or by the use of pumps to accelerate the process. The reinforcing effect is coupled with a drainage action, thus realizing a more effective stabilization at the face of tunnels in water-bearing ground (Zenti et al., 2012).

4.8.3 Summary

The implementation of truncated cone "umbrellas" of drainage pipes under the water table, when the designer considers it is best to advance under hydrodynamic conditions, creates a zone of high permeability around the cavity bringing down partially or fully the level of the water table and also, as a consequence, that of the hydraulic gradient. Since from a geomechanical viewpoint, the shear strength of the material is negatively affected by the presence of neutral pressures inside the pores and discontinuities,

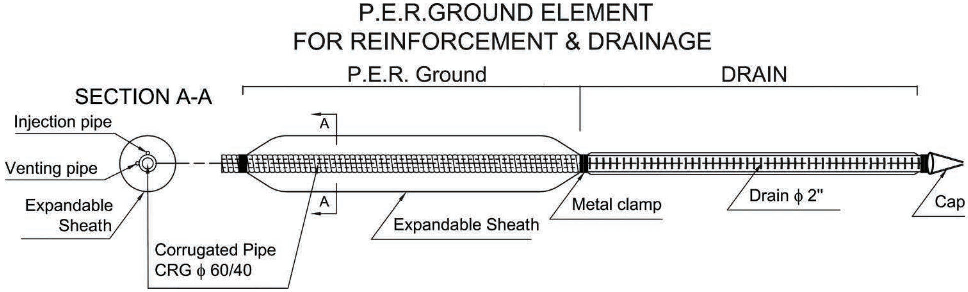


Figure 4.78 Innovative soil nailing systems for both reinforcement and drainage (Sterpi et al., 2013).

lowering these neutral pressures will improve the shear strength considerably. In order for this improvement to be effective for the band of ground around the tunnel and for all the ground within the advance core, these drainage pipes must be launched ahead of the face in a truncated cone "umbrella" configuration and they must always be positioned outside the core-face. If the amount of drainages installed at the boundary of the tunnel not adequately reduces the pore water pressure, it could be useful to install at the face some special reinforcing elements combining the reinforcing function with the drainage action.

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The chapter was developed as follows: Cassani was involved in chapter coordination and developing Sections 4.3, 4.5, 4.7 and 4.8; Gatti: Section 4.5; Zenti: Sections 4.3, 4.7 and 4.8; Manassero: Sections 4.4 and 4.6; Pelizza: Section 4.2; Pigorini: Section 4.1. All the authors contributed to chapter review. The editing was managed by Cassani and Zenti.

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